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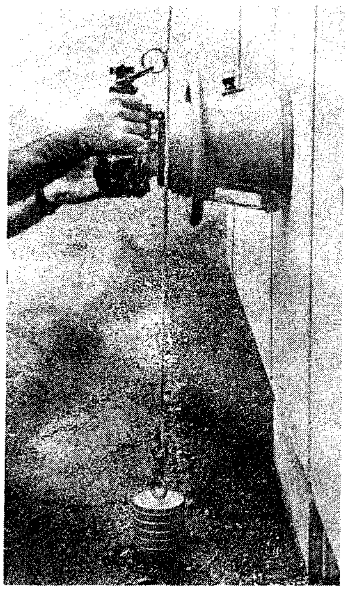

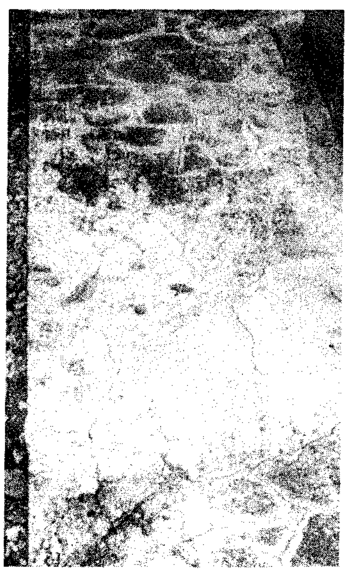

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REPAIR, EVALUATION, MAINTENANCE, AND REHABILITATION RESEARCH PROGRAM

TECHNICAL REPORT REMR-CS-9

INSPECTION OF THE ENGINEERING CONDITION OF UNDERWATER CONCRETE STRUCTURES

by

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GT	Geotechnical	EI	Environmental Impacts
HY	Hydraulics	OM	Operations Management
CO	Coastal		

COVER PHOTOS:

TOP — Underwater camera for inspection of structures in turbid water.

BOTTOM — Effect of alkali-silica reaction in concrete structures.

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All civil works structures must be continually evaluated for structural safety, stability, and operational adequacy. This is particularly true for underwater structures partially because it is more difficult to establish existing damages than it is for structures above the water; and partially because the underwater failure of many structures, such as dams, can produce very serious loss of lives and property. The overall objective of this report is to present organized information essential to the evaluation and monitoring of the engineering condition needed for continued safety of underwater civil works structures.

Specific objectives are: (a) to summarize the typical types of deterioration that occur in underwater concrete structures; (b) to provide engineering guidelines for inspecting the condition of underwater structures; and (c) to provide guidelines for interpreting inspection data.

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As recommended in the report, the inspection and evaluation of the engineering condition of underwater structures are preceded by a comprehensive review of the design of the structure and the construction techniques and materials used as well as its operation and maintenance history. Information is obtained from available engineering data on the structure and from onsite investigation. This report summarizes these activities, pertinent inspection procedures and techniques, and methods of evaluation used by the Corps of Engineers and the US Army Engineer Waterways Experiment Station. Methods and procedures that were developed by other agencies and individuals for the evaluation of concrete civil works structures are also included. Standardized and other methods that have proven satisfactory for inspection are emphasized. Methods that have potential for detection of the extent and cause of inadequacies in underwater concrete structures are also included.

PREFACE

This study was authorized by Headquarters, US Army Corps of Engineers (HQUSACE), under Civil Works Research Work Unit 32270, "Underwater Surveying." Mr. Henry T. Thornton, Jr., Concrete Technology Division (CTD), Structures Laboratory (SL), US Army Engineer Waterways Experiment Station (WES), was Principal Investigator. This work is part of the Concrete and Steel Structures problem area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program. Members of the Overview Committee of HQUSACE for the REMR Research Program are Mr. James E. Crews (DAEN-CWO-M) and Dr. Tony C. Liu (DAEN-ECE-D). Technical Monitor for this study was Dr. Liu.

Dr. Sandor Popovics, Professor of Civil Engineering, Drexel University, conducted the study and prepared this report under IPA Agreement No. 85-47c. The study was monitored at WES by Mr. Thornton and revised by Mr. Willie E. McDonald, SL, under the general supervision of Mr. Bryant Mather, Chief, SL, and Mr. John M. Scanlon, former Chief, CTD. Program Manager for the REMR Research Program is Mr. William F. McCleese, and Problem Area Leader for the Concrete and Steel Structures problem area is Mr. James E. McDonald, CTD. This report was edited and finalized for publication by Mmes. Gilda Miller and Chris Habeeb, editor and editorial assistant, respectively, Information Products Division, Information Technology Laboratory, WES.

COL Dwayne G. Lee, EN, is Commander and Director of WES, and Dr. Robert W. Whalin is Technical Director.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
inches	25.4	millimetres
pounds (force) per square inch	0.006894757	megapascals

INSPECTION OF THE ENGINEERING CONDITION OF UNDERWATER CONCRETE STRUCTURES

PART I: INTRODUCTION

Background

1. Engineering structures with failures or partial failures that might endanger the lives of the public or cause substantial property damage must be continually evaluated to establish the adequacy of structural safety and mode of operation. Such an evaluation can be performed through a proper inspection for the purpose of providing the information necessary to assess the condition (capacity, safety, extent and rate of deterioration, etc.) of a structure. Therefore, the usefulness of any inspection depends on the suitability and recording of the observations obtained for use in engineering evaluations. When evidence of concrete deterioration or distress is observed, the results of the evaluation are used for determining the proper remedial action, estimating the future service life or need for replacement, or rehabilitation planning purposes (Stowe and Thornton 1984).

2. Definite regulations have been outlined for continued evaluation of structures. For instance, US Congress Public Law 92-367 authorized the National Program of Inspection of Non-Federal Dams. More specific are Engineer Regulation (ER) 1110-2-100 (Headquarters, US Army Corps of Engineers (HQUSACE) 1983) and several technical guidelines by the American Concrete Institute (ACI) (ACI 1979a, b, and c, 1980). These regulations and guidelines are just as essential for underwater structures as they are for structures above the water level.

3. The complete evaluation of the condition of a structure is based on numerous factors such as design considerations, existing operating records and past inspections, condition surveys, maintenance repairs, in situ testing, instrumentation, identification of destructive phenomena, and final judgement concerning the state of the concrete (ACI Committee 207 1979b). This report is limited essentially to deterioration of concrete structures under water and practice in underwater inspection of concrete structures.

4. The evaluation of the condition of a concrete structure under water is much more complex than that of a structure above the water level. The reason for this is that before the structure can be evaluated and diagnosed, pertinent data and other information must be obtained for this evaluation. This latter activity is much more complicated when it is performed under water than when performed above the water, and it is more than putting together a team with the necessary technical expertise to evaluate the problem. Expertise is also needed in the underwater collection and recording of the pertinent information. It is possible, of course, for members of the technical team to collect the needed information for the evaluation, though highly unlikely. This is usually not practical because individuals having the technical background needed for reliable evaluation of a structure rarely are trained divers or operators of remotely operated underwater vehicles, and vice versa.

5. Therefore, in the case of an underwater structure, the technical team is usually supplemented with data collectors who are trained divers and/or operators of remotely operated underwater vehicles.

6. The procedures, equipment, and techniques discussed in this report overlap, in part, those found in an earlier report by Stowe and Thornton (1984) and in several other pertinent reports and publications. This was believed desirable to provide continuity to the subject and to permit this report to stand alone as an independent, self-contained document.

Objectives

7. The general objectives of a condition survey of a concrete structure in accordance with the American Society for Testing and Materials (ASTM C 823) are:

- a. To determine the ability of the concrete to perform satisfactorily under anticipated conditions of future service.
- b. To identify the processes or materials causing distress or failure.
- c. To discover conditions in the concrete that caused or contributed to satisfactory performance or to failure.
- d. To establish methods for repair or replacement without hazard of recurrence of the distress.
- e. To determine conformance with construction specification requirements.

f. To develop data to aid in fixing financial and legal responsibility for cases involving failure or unsatisfactory performance.

g. To evaluate the performance of the components used in the concrete.

In other words, such a survey is performed to determine whether the underwater structure meets design criteria under the prevailing service conditions; and the ability of the structure to perform satisfactorily in the future. Appendix B gives further details.

8. The objective of this report is to summarize in a self-contained document engineering guidance for the conduction of a condition survey of a concrete structure under water. The structural safety of the structure is the major concern and reason for conducting a condition survey.

Scope

9. This report focuses on techniques to inspect concrete in existing underwater structures or underwater portions of concrete structures. A general description of the planning and preparation required for an inspection program and methods of presenting the results are included in the report along with the description of the recommended inspection procedures. Primarily, the state of the art is presented as reported in the publications of the Corps of Engineers and other agencies. Techniques that have potential use in underwater inspection but which have not yet been completely developed for practical applications are also presented. The underlying principles, advantages, and limitations are discussed for each technique whenever such information was available.

10. The most thorough inspection of underwater concrete structures can be performed after unwatering (or dewatering) the structure. Unwatering, however, is not always possible, and even if technically possible, it is usually too expensive and may be politically sensitive. The inspection is performed along the same guidelines as the inspection of traditional, above-the-water concrete structures (Stowe and Thornton 1984). Therefore, this report considers inspection of underwater structures only.

11. The underwater structure must be evaluated in view of its specific nature and site conditions and with the full awareness of the many variables involved, some of which may not be precisely known.

12. A condition survey may be limited only to isolated areas displaying deterioration; or the investigation may be concerned with general distress, such as excessive deflection or collapse of structural members. It may involve study of the dislocation of entire structures or large portions of structures. The investigation may be confined chiefly to the study of the concrete or it may require substantial research into other circumstances, such as foundation conditions, conditions of service, construction practices, and comparisons with other structures (ASTM C 823).

Levels of Inspection

13. A Navy technique manual (HAN-Padron Associates and Naval Civil Engineering Laboratory (NCEL) 1984) recognizes three basic levels of inspection of underwater and marine facilities. They are distinguished by the resources and preparation needed to do the work and the type of damage/defect that is detectable, as follows:

- a. Level I - general visual inspection. This type of inspection does not involve cleaning of any structural elements and can, therefore, be conducted much more rapidly than the other types of inspection. The purpose of the Level I inspection is to confirm as-built structural plans, provide initial input for an inspection strategy, and detect obvious damages.
- b. Level II - close-up visual inspection. This type of inspection will generally involve prior or concurrent cleaning of part of the structural elements. The purpose of Level II inspection is to detect surface damage that may be hidden by marine growth. Since the cleaning process will make this type of inspection more time consuming than the Level I inspection, it will generally be restricted to the critical areas of the structure.
- c. Level III - nondestructive testing. This type of inspection will be conducted to detect hidden or beginning damage. The training, cleaning, and testing requirements will vary depending on the type of defect/damage that is anticipated and the type of inspection equipment used. In general, however, the equipment and test procedures will be more sophisticated and require considerably more experience and training than either the Level I or Level II inspection.

Table 1 lists the defects that are detectable with the three types of inspection.

Table 1

Level of Inspection Versus Detectable Damage to Underwater Concrete Structures (after HAN-Padron and NCEL 1984)

Level	Purpose	Detectable Damage
I	Confirm as-built condition and detect severe damage	Severe damage (freeze/thaw, erosion, cavitation)
II	Detect surface defects normally obscured by marine growth, silt, or other debris	Surface cracking Deterioration of concrete due to sulphate action (moderate freeze/thaw, moderate erosion, moderate cavitation) Severe corrosion of rebar Spalling of concrete surface
III	Detect hidden and beginning damage	Location of rebar Beginning corrosion of rebar Internal voids Change in material strength

14. The level of inspection to be used for a particular task must be decided early in the planning phase. Often, the requirements of the local public works office or other authority will dictate the level of inspection.

15. The time and equipment required to carry out the three different levels of inspection are quite different. The time required for any particular level will depend on a number of factors including visibility, currents, wave action, water depth, severity of marine growth, and the skill and experience of the personnel. For instance, a rough estimate is that under moderate conditions a Level I inspection of a 12-in.-wide strip of concrete sheet pile will require 3 min, whereas a Level II inspection will require 15 min.

Underwater Inspection With and Without Diver(s)

16. Underwater inspection may include the use of one or more divers or a remotely operated vehicle (ROV). ROV's used for the types of inspections

described in this report usually have video cameras and lights installed and are controlled from the surface with little diver participation. They can range from a small, relatively inexpensive system to a highly capable but expensive system.

17. The advantage of using ROV systems is that ROV's can compensate for the inherent limitations of diver systems under water because they can have very deep operating depths, long operating duration, can repeatedly perform the same mission with no performance degradation, and can be operated in locations where the water currents and tidal conditions make use of divers unacceptable. Disadvantages of using ROV systems are:

- a. ROV inspections are usually expensive because they require large support no matter what depth of operation the mission requires. Such support consists of power generators, display monitors, vehicle controllers, cables, and spare parts.
- b. ROV's are usually less flexible and less reliable.
- c. ROV systems tend to have greater maintenance requirements than does the diver system.
- d. Video cameras installed on ROV's may distort angles and dimensions, and extent of deterioration or damages when they are not referenced.

18. Inspection with a ROV system depends not only on the nature of the underwater activity but also on the type of ROV. Therefore, details of such an inspection procedure should be worked out for each case in consultation with the manufacturer and/or operator of the ROV in question. Information on ROV's, including a general description of the various types is given in a recent report prepared by the Marine Technology Society (MTS) Subcommittee on Remotely Operated Vehicles (MTS 1984). There are towed vehicles, bottom-crawlers, vehicles which carry their own power supply, and those which are controlled remotely from the surface. Though all are not tested, six basic types of ROV's are:

- a. Tethered, free-swimming. This type of vehicle constitutes the vast majority of ROV's. All have closed circuit television (CCTV) cameras, maneuverability in three-dimensions, and are cableconnected to a surface or subsurface platform from which they are controlled and to which they transmit their data and observations. Most of these vehicles receive their power from the support platform, but a significant number are powered by batteries which are carried onboard. All vehicles of this type are designed for midwater operation, as opposed to ROV's which are designed to operate in contact with the bottom. They are generally positively buoyant and rely upon a vertical

thruster to descend. Many vehicles are equipped with mechanical manipulators to perform a variety of work and to deploy instruments. This type of vehicle is part of a system which consists of: a power source that is ship's power or a dedicated generator; a control/display console used to monitor the vehicle's condition and through which the vehicle is controlled; a handling system that launches and retrieves the vehicle and manages its umbilical, a cage or launcher within which the vehicle is launched/ retrieved and from which it is deployed at the working depth and to which a short tether is attached (this cage component is optional); and, finally, the vehicle itself. Recent variants to the foregoing description are towed, midwater vehicles that have the added capability of maneuvering in three dimensions, much like the tethered, free-swimming or bottom-crawling vehicles. The Remote Underwater Manipulator (RUM III), for example, operates in a conventionally towed mode, but can also operate in a bottom-crawling mode to perform selective sampling or detailed manipulative work. The Towed, Unmanned Submersible (TUMS) operates in the towed configuration and can, when the need to stop and perform detailed investigations arises, operate as a tethered, free-swimming vehicle by employing the onboard thrusters. The majority of vehicles in this wide category have design operating depths ranging from 100 to 10,000 ft, weight in air ranging from 70 to 12,000 lb, and dimensions ranging from basketball size to that of a small automobile. An operating and maintenance crew of from one to ten is required, depending upon the complexity of the vehicle and nature of the job, but generally, a crew of three suffices.

b. Towed:

(1) Midwater. These vehicles are propelled and generally powered by a surface ship via a surface connected cable. CCTV (real time or slow scan) and photographic systems are carried. They are designed to operate in midwater, but may have the capability to make periodic contact with the bottom. Maneuverability in the X and Y axes (horizontal) is controlled by the heading of the support ship, the Z axis (vertical) is controlled by reeling in/reeling out cable. Vehicles in this category are generally multi-instrumented and are used for broad area reconnaissance and mapping, mineral and geological surveys, and search, identification, and location of objects of interest. Most of the vehicles are designed for operation in water depths of 20,000 ft. Vehicles of this type are designed primarily for long range, long duration, broad area, search, location, survey, and inspection.

(2) Bottom- and structurally reliant. Propulsion and power is supplied by the towing ship, and a CCTV camera is generally carried. Vehicles are designed to be towed in contact with the bottom (bottom-reliant) or are supported by and in contact with a pipeline (structurally reliant). All vehicles in this category are one-of-a-kind and

purpose-designed. They are all massive affairs which are negatively buoyant by several tons, although some have the capability of adjusting buoyancy to a positive condition when submerged. All of this type are used for cable or pipeline burial.

c. Bottom-reliant. All power and control for this type vehicle is from the surface support vessel. CCTV cameras are carried to observe and document the work in progress. Vehicles obtain propulsion from wheels or tracks in contact with the bottom. They are generally large, massive affairs with no easily definable geometry which may also adjust buoyancy to a negative, neutral, or positive condition. Almost all of these vehicles are purpose-built to perform a single task, such as pipeline/cable trenching, bottom excavation, maintenance, inspection, soil investigation, pipeline backfilling, or nodule collection.

d. Structurally reliant. Vehicles in this category also obtain power and control from the surface. Propulsion is obtained from wheels, tracks, or push-pull rams in contact with a structure, although some midwater capability often exists for transit to/from the structure. Most have CCTV, and all are designed to perform a single task, such as pipeline trenching, oil tank sounding, ship's hull cleaning and inspection, and Subsea Production System (SPS) maintenance and inspection.

e. Untethered (autonomous). These vehicles are self-powered and operate without physical connection to the surface (these features sometimes give rise to the term "autonomous"). Maneuverability is generally three-dimensional and data collected are stored aboard the vehicle. None, at present, transmit TV signals through water to the surface and all are generally in the developmental stage. They may operate within a preprogrammed schedule or, in some instances, they may receive course and altitude change commands from the surface via an acoustic link. Operating depth ranges from 100 to 20,000 ft and dive duration is presently from 4 to 6 hr.

f. Hybrid. Hybrid vehicles are relatively recent participants in the undersea vehicle field. They constitute combinations of ROV's that are remotely controlled as well as directly controlled in situ by the diver or pilot. The tasks these vehicles perform are as follows: pipeline trenching (diver-controlled, tracked, bottom-crawler); diver assistance (diver- and/or surface-controlled, tethered, free-swimming); structure inspection and maintenance (pilot- and/or surface-controlled, tethered, free-swimming); pipeline anchoring (diver- and/or surface-controlled, tracked bottom-crawler), and cable burial (pilot-controlled, tracked bottom-crawler). There are other types of hybrids which always operate as ROV's, but operate in a variety of modes.

The primary distinction between these groups is the means whereby they obtain power for operation (MTS 1984). The ROV will usually be equipped with CCTV,

and it is capable of accommodating some types of mechanical manipulators. Surface cleaning equipment and selected types of nondestructive testing apparatus have been operated from ROV's. Further details concerning applications, task considerations, environmental aspects, supporting systems, structural aspects of the vehicles, tools and sensors, personnel, operational and navigational considerations, etc. can be found in the MTS Subcommittee report as well as in the literature (MTS 1985). The remainder of this part of this report will focus on underwater inspection with diver(s).

19. The requirements of the specific survey or inspection mission dictate the most efficient, cost effective, and safest system to be deployed.

20. Diver inspection can be conducted by a free-swimming scuba diver or surface-supplied diver. Advantages of using the diver system for underwater inspection are:

a. It is a versatile and flexible system.

b. It is simple, requiring minimum support, especially the scuba diver in shallow water application.

c. In most cases it is relatively inexpensive.

21. The disadvantages of using diver(s) for underwater inspection are:

a. Diving is severely depth limited. The extent of support needed (ship, medical help, decompression chamber, etc.) and number of support personnel increases rapidly with the depth of diving.

b. The diver has severe depth/time limitation for even shallow depths. For instance, the maximum time allowed for a 90-ft, no-decompression dive is 30 min.

c. The diver is limited in his number of repeated dives, even for moderate depths. This is especially severe for depths greater than 100 ft.

d. The diver's visual, auditory, tactile, and spatial perceptions are different under water than in air. Therefore, he is susceptible in making errors in observations and recording data.

e. Cold temperatures decrease the diver's ability to concentrate. Consequently, information about the overall underwater situation is seldom observed or retained.

Inspection Team

22. Each agency should evaluate its underwater inspection requirements before selecting a method for performing these inspections. The three methods currently in practice are:

a. Contract divers.

b. In-house staff/divers.

c. Combination of in-house and contract divers.

23. The level of inspection and the staffing for underwater inspection differ among agencies. The number of structures with major underwater substructure components is a factor in an agency's decision to use in-house or contract inspection. Some of the coastal-area states have staffed and trained their own underwater teams for both routine and emergency work. Other coastal-area states depend on contract inspection for all underwater work.

24. When the inspectors are agency employees, the required education, experience, and physical requirements are clearly defined. However, when the competence of the underwater inspection crew is the responsibility of a contractor, the required qualifications of the inspectors are not always specifically defined. It is common practice to require that contract divers be certified. Organizations that certify divers include the Professional Association of Diving Instructors, National Association of Underwater Instructors, and the American Diving Contractors Association. EM 385-1-1 (HQUSACE 1984) and ER 385-1-86 (HQUSACE 1982) standards should be adhered to on all Corps diving projects.

25. With proper education, training, experience, equipment, and agency support, both agency and contract dive teams should be capable of performing the inspection tasks.

26. According to Lamberton et al. (1981), the advantages of contracting for all underwater inspections include:

a. Elimination of the need to recruit, train, and maintain diving teams.

b. Reduced chance of equipment being idle when the team is not actively involved in diving operations.

c. Opportunity to specify a range of work, qualification, equipment, etc., in each contract.

d. Elimination of problem associated with assignment of divers to other work.

27. The disadvantages of contracting for all underwater inspections include:

a. Inability to respond quickly to emergency situations.

b. Problems associated with bidding for engineering services.

c. Possibility of new contracts each year.

d. Assignment of final responsibility for the engineering report of the inspection.

28. Among the advantages of using an in-house dive team for all inspections are:

a. Total control of the inspection efforts.

b. Ability to train inspectors in specific problems of design, construction, and materials.

c. Use of dive team for construction inspection.

d. Accumulation of experience and knowledge gained in repeated dives during scheduled inspections.

29. The disadvantages include:

a. Capability of dive team may be exceeded in some cases.

b. Need for alternate work assignments if dive team is not multiqualified.

c. Pressing normal work assignments (nondiving) conflict with scheduling of dives or duration of diving inspections.

30. Perhaps the best situation exists when an agency has limited dive capability for routine, construction, and emergency inspections with a backup contract for additional diving. In some cases, agency divers can work with or direct contract divers.

31. When requesting bids on contract diving, the qualifications of the divers, the size and amount of special equipment, and the rate of renumeration must be considered (Lamberton et al. 1981).

32. There are agencies which maintain the policy that divers should be well-trained in structural inspection. Nevertheless, the use of underwater video cameras, video tape, photography, etc. has permitted agencies to employ nonengineer divers with increased confidence. In this way it is possible to divide the work into two parts, namely the diver/inspector who makes and records observations which, in turn, are used by an engineer who makes the engineering assessment. For example, the diver may observe, measure, and report that a concrete wall has a hole measuring 2 by 3-1/2 ft at depth of 1-1/2 ft below the water level and that behind the hole is a cavity 6 ft deep. The diver should not report that the wall has diminished structural capacity, or that it is unsafe, etc. (Brackett, Nordell, and Rail 1982). Even when a qualified engineer conducts the underwater inspection himself as a diver, the complex interaction of the individual structural elements precludes any

realistic on-the-spot assessment of the overall structural capacity based on localized defects. Buehring (1981) states:

> "The engineer trained to dive can become more familiar with the underwater environment and can also obtain pertinent information first hand. However, by assuming the role of diver/inspector, the engineer may actually be increasing his inspection risk by working in an environment that may affect his mental state. In zero visibility, it may be necessary to inspect by touch alone, in which case the engineer's accuracy may be no better than that of the commercial diver. The professional engineer who dives is likely to make more judgmental decisions than the commercial diver. Also because the engineer's authority exceeds that of the commercial diver, his observations and conclusions are more likely to be accepted at face value, even though they may not always be accurate."

33. A Technical Note prepared by NCEL also makes the following observation (Brackett, Nordell, and Rail 1982):

> "Since there is a limited supply of diver trained engineer/inspectors and the possible increased inspection risk of employing engineer divers to conduct the underwater inspection, the initial philosophy adopted under the Specialized Inspection Systems (SPINS) R&D [Research and Development] Program is to develop techniques and procedures which require a minimum of specialized training or background for the underwater inspector (although specialized or advanced training will most likely be required for a small number of topside personnel). This training should enable the diver to recognize distress and deterioration, including incipient problems so that preventive actions can be taken. The applicability of this philosophy can only be determined after development of the individual techniques and assessment during actual inspections."

34. Regardless of the final training or experience to be required for the underwater inspectors, it is important that the underwater inspector report deviations from the assumed as-designed or as-built condition. The diver-inspector should report:

- a. Changes in the geometry (cross sections, lengths, etc.) of structural elements.
- b. Changes in material properties (such as variations in concrete surface).
- c. The presence of defects.

35. It is of little value to the facilities engineer, when making a structural condition assessment or developing a maintenance or repair project, to have an inspection report for various structural elements filled with inspection data such as severe deterioration, moderate defect, or catastrophic failure.

36. Since the inspection team is reporting the structural condition and is not making an engineering assessment, the team should be accompanied by a facilities engineer or engineers. The engineer can provide guidance to the inspection team during the inspection and can make preliminary engineering assessment decisions on the spot, allowing the inspection to progress much more quickly, efficiently, and accurately than would be possible if he were to make the assessment using only the written reports provided after completion of the inspection.

37. The suitable sizes and compositions of the two teams depend on the problem to be investigated. In most cases, a technical team of four members or less is sufficient. It is usually desirable for team members to have backgrounds in civil and mechanical engineering and engineering geology, as well as in design, construction, maintenance, and operation of underwater concrete structures (Stowe and Thornton 1984).

38. Safe and efficient diving equipment is an important factor in underwater inspection. A good description of some of this equipment is presented by Lamberton et al. (1981).

PART II: DETERIORATION OF UNDERWATER CONCRETE STRUCTURES

Properties of Concrete

39. Hardened portland cement paste, mortar, and concrete appear to be rock-like materials, and indeed many properties of the concrete (and mortar) are similar to rock properties. One of the few exceptions is the chemical resistance because, as a rule, hardened concrete is more susceptible to chemical attacks than rocks due to the susceptibility of the hardened cement paste.

40. Furthermore, since a chemical must be present in liquid or vapor form to attack concrete, concretes in contact with water are more susceptible than concrete which is not in contact with water.

41. Most concrete will perform satisfactorily when exposed to various atmospheric conditions, to most waters and soils, to many kinds of industrial wastes, and other chemicals. Most concrete deterioration starts as a result of error(s), such as poor construction design, improper concrete composition, poor construction technique, and/or inadequate inspection follow-up and quality control. There are, however, some environments under which the useful life of even the best concrete will be short. The technical term for the reduced quality of concrete is "deterioration."

42. In principle, a concrete can deteriorate because its cement paste deteriorates, because its aggregate deteriorates, or both. In practice, however, almost always the attack on the hardened cement paste is the main cause of deterioration due to the better chemical resistance of the aggregates. Even when the aggregate is attacked by the chemical, the damage to the cement paste is usually more severe. "It is always easier to prevent concrete deterioration than to repair it" is a frequently repeated adage.

General Aspects of Concrete Deterioration

43. The term "durability" refers to the capability of a material to withstand the conditions for which it was designed without significant damage during its service life. An antonym to durability is "deterioration." The deterioration of a concrete is usually initiated by chemical processes, although physical and mechanical factors can also cause deterioration alone or in combination with chemical agents.

44. The nature and magnitude of the effects of the chemical agents on the deterioration of hardened cement paste and concrete depend on both the chemical composition and the internal structure of the hardened cement paste (Popovics 1979).

45. The attacking chemical must be above a certain threshold concentration in the liquid to be aggressive, that is, to produce significant deterioration. However, a chemical in a given concentration can produce damages of different magnitude depending on the other prevailing circumstances. Some of the other factors that reduce the ability of cement paste, or concrete, to resist deterioration are as follows (Highway Research Board 1970):

- a. Higher porosity in concrete resulting from inadequate proportioning, poor compaction, high water-cement ratio, or a combination of these.
- b. Higher permeability and higher absorption of the fluid by the concrete as a consequence of the higher porosity.
- c. Improper cement type (in some circumstances).
- d. Poor curing of the concrete.
- e. Alternate wetting and drying.
- f. Increased fluid velocity.
- g. Replenishment of the attacking chemical agents.
- h. Higher temperatures.
- i. Corrosion of reinforcing steel.

46. Note that concrete which is subjected to chemical agents on one side only is more vulnerable than otherwise, because the chemical agents in the fluid can migrate through the concrete more easily, bringing more damaging material into contact with the cement paste. In addition, thin or small structural elements are more sensitive to aggressive attacks than massive elements, again because of the increased migration of the chemical agents through the concrete. For instance, large foundations in aggressive ground water were found intact after many years of service, whereas considerable deterioration was observed in concrete pipes in the same area. Kleinlogel (1950) also reports that concrete structures of large dimensions withstood corrosive actions of seawater much better than small laboratory specimens made from the same concrete. Finally, the damage to concrete is generally most severe in the splash and tidal zones. The different exposure zones are defined in Figure 1.

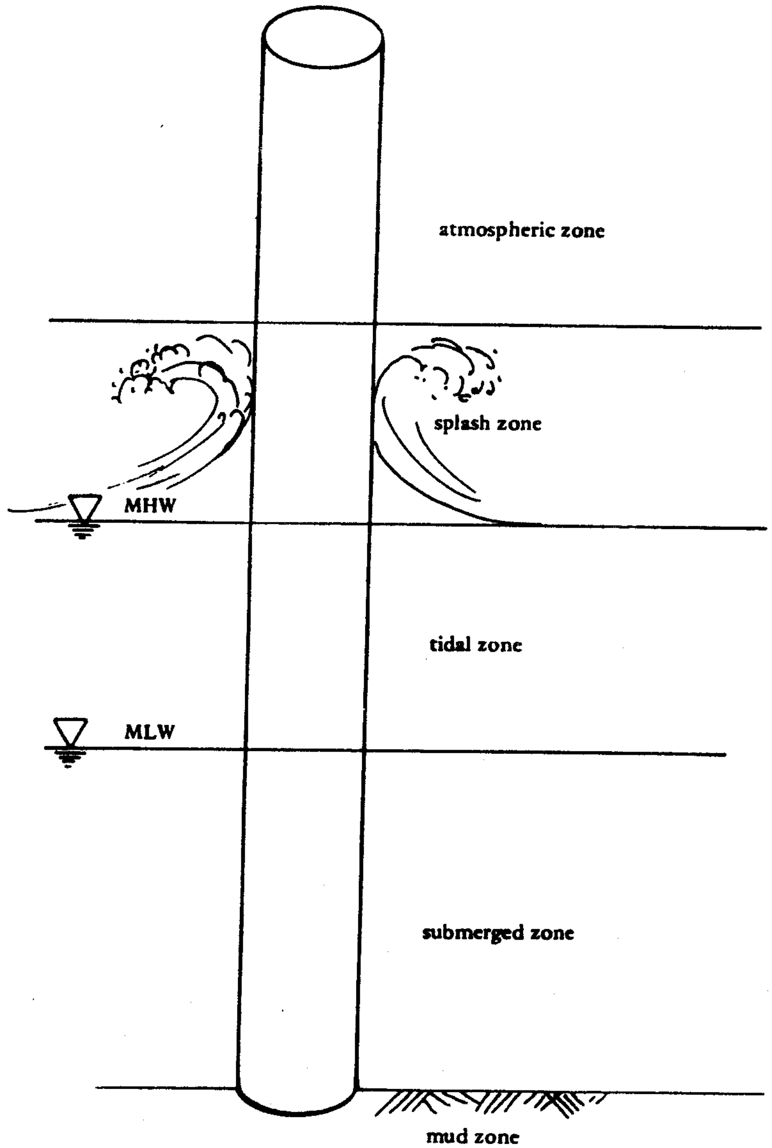

Figure 1. Exposure zones (from Brackett, Nordell, and Rail 1982)

47. Some of the various mechanisms associated with deterioration of concrete are briefly described (Popovics 1985):

a. Leaching. Deterioration due to leaching is caused by water containing carbonic acid or having low carbonate hardness. In this process the calcium hydroxide developed by the hydration of portland cement is leached out. The deterioration does not reach large proportions unless the dissolving power of the water is high and the concrete is porous, but it can produce unsightly efflorescence (Figure 2).

Figure 2. Efflorescence produced by the leaching of concrete

b. Nonacidic reaction. Deterioration consists of nonacidic reactions between the calcium compounds of the hardened cement paste and compounds of the aggressive water. Here the reaction products are weaker than the original hydration products and/or have the tendency for leaching. Thus, the process weakens the concrete and makes it more porous. The mechanism can be base exchanged as in the case of magnesium salts. Other examples can be the saponification reaction between animal fats and calcium compounds or the reactions of sugars forming calcium saccharate.

c. Acidic reaction. Deterioration consisting of reactions involving acidic water which results typically in the decomposition of the calcium compounds and hydration products of the cement may also be encountered. In this process, the hardened paste is gradually dissolved by the acid. Even if the aggregate particles are acid resistant, the paste can no longer hold them together and a gradual ravelling takes place resulting in disintegration of the concrete.

d. Sulfate/alkali attack. Deterioration consists of reactions accompanied by excessive expansion. These can be reactions between the hydrated aluminate compounds and salts, mostly water solutions of sulfate, producing the so-called sulfate attack. In such cases, calcium sulfoaluminate crystals form which may have cementing capability. Nevertheless, the continuous growth of these crystals induces excessive internal stresses producing cracks and gradual disintegration of the concrete. Another frequent case is the aggregate-alkali reaction that takes place in the presence of water between the alkalies in portland cement and certain rocks and minerals containing active siliceous or carbonate materials (Popovics 1979). These again result in large volume increases accompanied by excessive expansion of the concrete, cracking, gelatinous discharge, spalling, and chalky surfaces or rings around aggregate particles (Figure 3).

Figure 3. Effect of alkali-silica reaction in concrete structures

e. Physical processes. Deterioration is caused by physical processes. A typical such process is when a salt solution

penetrates concrete continuously, becomes concentrated at an evaporating face, and subsequently the salt crystallizes. The crystals can appear on the concrete surfaces as efflorescence, and/or fill up pores in the concrete developing excessive internal pressure in the concrete and causing spalling and cracking. A special case of this is frost damage in which the penetrating liquid is water; pressure is developed when the water freezes and results in the progressive formation of a series of fine random cracks at close intervals on a concrete surface, or D-cracking damage (Figure 4).

Figure 4. Typical D-cracking damage in concrete caused by freezing and thawing

f. Mechanical processes. Deterioration is mechanical in nature and consists essentially of abrasion which is caused by waves, or moving solid particles in water or air, or by just ordinary wear and tear; and cracking from excessive shrinkage, restraining uneven thermal expansion, overload, and repeated loading, etc. (Figure 5).

48. Some of these types of deterioration are more likely to occur in nature, others in industrial plants and sewers. In any case more than one class can, and usually do, occur simultaneously or consecutively in a deterioration process. Usually a specific type of deterioration predominates.

49. Several specific examples of important cases of underwater deterioration of concrete are presented in the following paragraphs.

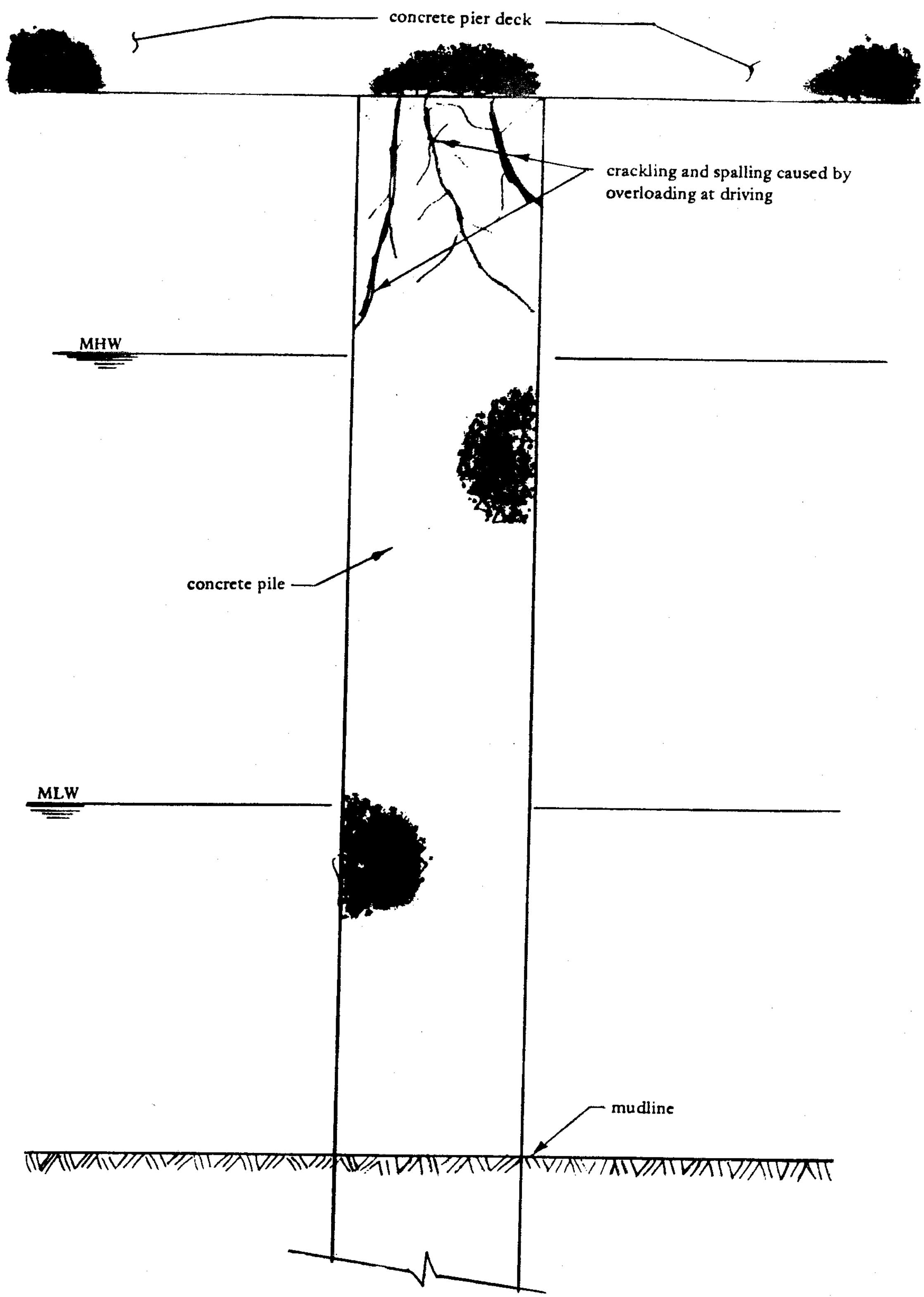

Figure 5. Mechanical deterioration of a concrete pile (from Brackett, Nordell, and Rail 1982)

Erosion

50. According to ASTM G 40-82 (ASTM 1982) the term "erosion" is defined as progressive loss of original material from a solid surface due to mechanical interaction between the surface and a fluid, a multicomponent fluid, impinging liquid, or solid particles. Because of the broad scope of this term, it is recommended that the term be qualified to indicate the relevant mechanism, as for instance impingement erosion, abrasive erosion, etc.

51. The abrasion or erosion resistance of a concrete is controlled by the same factors, although not necessarily to the same extent, as its compressive strength. These factors are: the quality of matrix mostly cement paste; the aggregate; and the strength of the interface on matrix and aggregate particles. Although concrete ingredients usually have high compressive strengths, even the best concrete that can be made under practical circumstances cannot withstand the forces of severe abrasion or cavitation for any prolonged period of time.

52. The action of the abrasive particles in the water is controlled largely by the velocity of water, the abrasive material, the general surrounding conditions, and the angle of impact. When the relative motion of the solid particles is nearly normal to the solid surface, the water is called impact erosion or impingement erosion. When this relative motion is nearly parallel to the solid surface, the wear is called abrasive erosion.

53. These various actions produce different types of erosion. For instance, in the case of an ideally pure impingement erosion, the matrix is gradually displaced by the repeated impacts, the deteriorated matrix cannot hold the coarse aggregate particles, and the resulting ravelling can produce localized, deep cavitations in the concrete. This mechanism indicates that the resistance to this type of erosion is controlled mainly by the softer component of the concrete, which is usually the matrix. In contrast, a pure abrasive erosion is less localized. The wear is distributed more or less uniformly producing a relatively smooth concrete surface. This type of erosion is more sensitive to the hardness of the coarse aggregate. In most cases, however, the two types of mechanisms act simultaneously, so the character of such erosion will be a combination of the impingement and abrasion erosions with one or the other dominating once the surface paste is worn through. Such combined erosion produces typically the so-called "differential

wear," where the worn surface is uneven, with the harder component protruding from the softer material.

54. Two major erosion types occur in underwater structures, as follows (Prior 1966):

a. Wear on hydraulic structures, such as dams, spillways, bridge abutments, and tunnels due to the action of abrasive materials carried by waters flowing at low velocities (attrition plus scraping). This type of wear is primarily a cutting action. The abrasive erosion of concrete by silt, sand, gravel, and other solids can be quite severe. Stilling basins which are not self-cleaning and in which rock and sand collect are eroded by the movement of the solids by eddy currents in the pool (Figure 6). Similarly, concrete over which large quantities of sand and gravel are transported by floods may be eroded seriously.

b. Wear on concrete dams, spillways, tunnels, and other water-carrying systems where a high hydraulic gradient is present. This is generally known as cavitation erosion as distinguished from abrasive erosion. Cavitation is completely an impact abrasion. It is caused by the abrupt change in direction and velocity of a liquid to such a degree that the pressure at some point is reduced to the vapor pressure of the liquid. The vapor pockets so created collapse with a great impact upon entering areas of high pressure, which eventually causes pits or holes in the concrete surface. Also, particles torn loose by this action continue to add to the abrasion problem by causing further wear.

55. Damage resulting from cavitation is not common in open conduits at water velocities below 40 fps. However, concrete in closed conduits has been pitted by cavitation at velocities as low as 25 fps where the air pressure was reduced by the sweep of the flowing water. At higher velocities, the forces of cavitation are sufficient to erode large quantities of high-quality concrete in a comparatively short time. This erosion can be minimized, to some extent, by careful attention to form alignment and avoidance of rough uneven surfaces (Prior 1966).

56. Further details of some of these erosion types are given in the report of ACI Committee 210 (1979c).

57. Erosion damages occur frequently in underwater structures. A typical example is the erosion observed in the stilling basin of the Chief Joseph Dam on the Columbia River in Washington. This stilling basin is approximately 920 ft wide and 220 ft long, and is divided into four rows of concrete slabs approximately 65 ft wide, 50 ft long, and 5 ft thick. They are anchored to the foundation rock with grouted anchor bars. The slabs are unreinforced

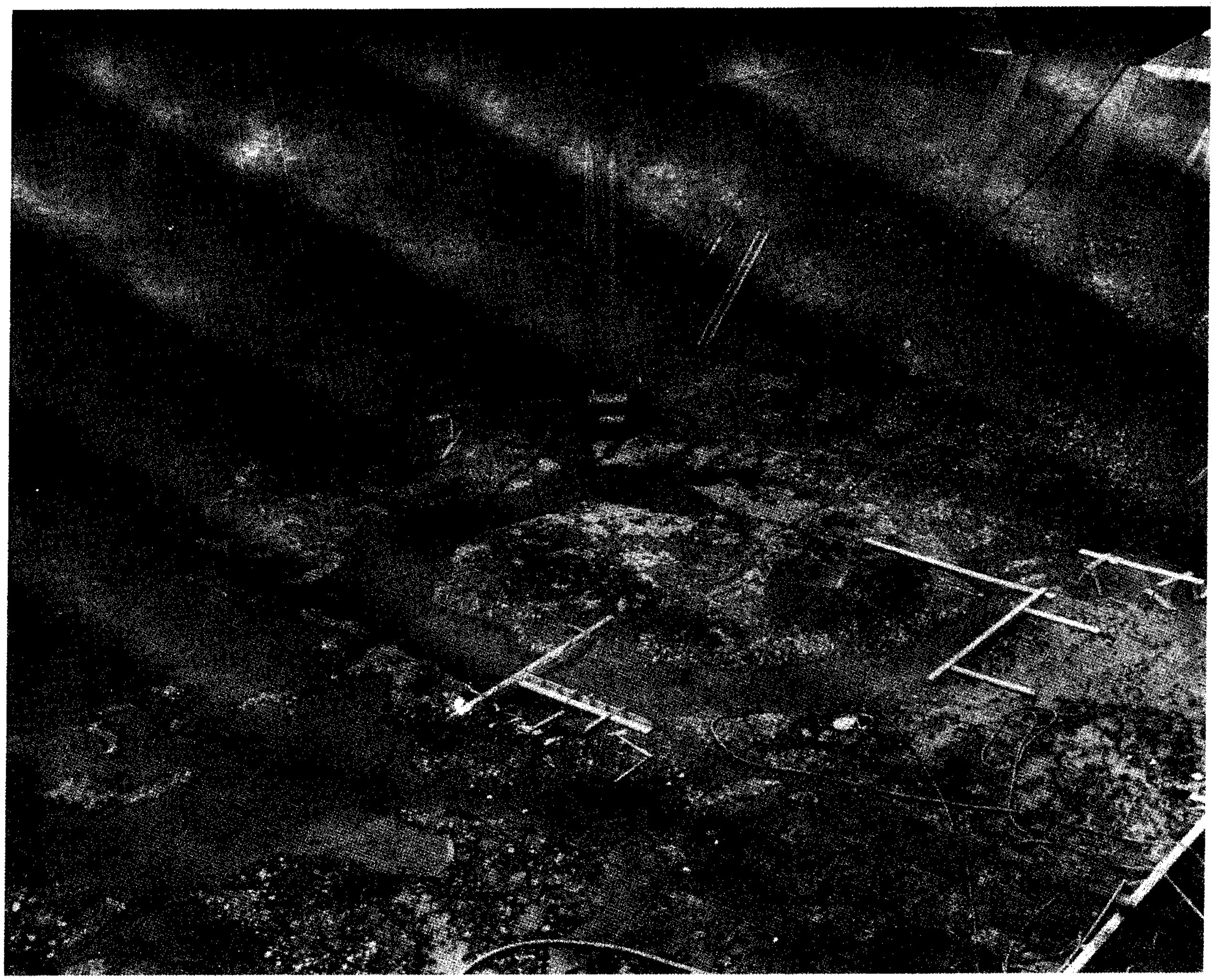

Figure 6. Abrasion erosion of a stilling basin slab, Kinzua Dam (McDonald 1980)

except the downstream row, which carries a single row of baffles and the end sill.

58. Extensive areas of eroded concrete were discovered during an underwater inspection in March 1957, 2 years after the project became operative. An underwater survey during November 1957 substantiated and located the areas of widespread damage. Areas were found in which there was erosion of concrete in excess of 12 in. in the slab and in the baffles, which exposed the reinforcing steel. Severe damage was concentrated in localized areas immediately upstream of some baffles, between baffles, and between baffles and the end sill. Two small scour holes approximately 5 ft in depth were discovered

between the baffles and end sill. Additional damage to the stilling basin, observed during the March 1960 survey, was found to be minor as compared with the pattern of major damage found in the 1957 survey.

59. The major damage was apparently caused by abrasion of large-size rock particles together with concentration of river flows through constricted sections of the stilling basin during the dam construction. The stilling basin has been subjected to passage of flood flows that cause velocities in excess of 100 fps at the bucket of the spillway. Under such circumstances, progressive deterioration from flow impingement and cavitation of the damaged and adjacent areas could be expected.

60. Incidentally, the stilling basin was repaired with an underwater method by placing overlaying slabs on the eroded surface (McDonald 1980).

61. Since there are different types of erosion, no single test method can be satisfactory for all conditions. One method used to determine the relative resistance of concrete surfaces to abrasion-erosion under water is standard test method CRD-C-63 (US Army Engineer Waterways Experiment Station (USAEWES) 1949). This test method involves the use of a rotating device, such as a drill press, capable of holding and rotating an agitation paddle at a speed of 1,200 ± 100 rpm. The test specimen is placed inside a steel container of specified dimensions. The container is partially filled with water, and steel balls of varying diameters are also placed in the container. Rotation of the agitation paddle causes swirling of the water and the steel balls, which simulates an abrasive action, acting upon the test specimen similar to abrasion-erosion actions experienced by concrete structures under water. This test method (Figure 7) should provide useful information on material and mixture selections. A comparison of various methods to test the abrasion resistance of concrete is presented by Lane (1978).

Attacks by Seawater on Concrete and Reinforced Concrete

Concrete

62. The durability of concrete in seawater can be reduced by several classes of deterioration. More specifically, concrete in seawater can be deteriorated by chemical factors, freezing and thawing, erosion, corrosion of the reinforcement, and crystallization of salts due to their concentration

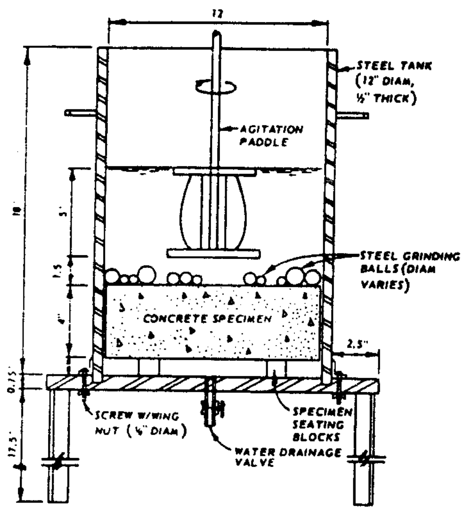

Figure 7. Test apparatus (inch-pound units) (USAEWES 1949)

being increased by capillary action and evaporation, particularly in and above the tidal zone.

63. Regourd (1980) summarized the physicochemical effects of seawater on hydrated cement, as follows:

a. The chemical attack of seawater on hardened cement paste occurs only in the case of permeable concretes (low cement content, high water-cement ratio, insufficient impermeability). A high porosity of the concrete aids in the diffusion of the aggressive ions, Cl^-, SO_4^{2-}, Mg^{2+}, which results in a series of chemical reactions, leading to the deterioration (erosion, cracking) and finally the destruction of concrete.

b. C_4AF, in contrast to C_3A, does not act deleteriously. Though it forms ettringite, this through-solution ettringite is more dispersed and needles of Fe-substituted trisulfoaluminate never grow very large. With C_3A, the expansive ettringite is more localized, gathering around the aluminate grains because of the very large supersaturation of the solution in aluminia. C_6A_2F, richer in alumina than C_4AF, is less desirable.

c. Portland cements with lower than 10 percent C_3A, such as Type II and especially Type V cements, are resistant to deterioration in seawater. An amount of C_3A higher than 10 percent leads to a greater expansion in cements rich in C_3S. Kuenning (1966) also found that Type V cement in hot seawater appeared to be more resistant to attack than those immersed in seawater at 73° F (23° C).

d. Cements containing more than 65 percent slag have the greatest resistance to deterioration in seawater.

e. The stability of cements containing 20 percent pozzolan

depends upon the mineralogical composition and the reactivity of the pozzolan. This stability in seawater is not always directly related to the lime absorption of the pozzolan.

f. Compressive or flexural strengths do not serve as a good basis for measuring concrete durability once deterioration starts. The measure of expansion seems better suited for the evaluation of the deterioration.

64. All this indicates that portland cement concretes deteriorate in seawater, although this process is slow if the concrete is dense enough.

Reinforced concrete

65. Steel reinforcement may cause serious durability problems in concrete under special circumstances. Instances of distress due to steel corrosion in concrete might be found in many applications of reinforced concrete: buildings, slabs, beams, piles, tanks, pipes, etc. It is particularly frequent and serious when not only water and oxygen but also chloride ions can reach the reinforcement causing the so-called "chloride corrosion." Chloride can be in the concrete (a) because it was added to the concrete as an admixture, or inadvertently as an impurity in the aggregate or in mixing water, such as seawater; or (b) because chloride solution penetrated the hardened concrete. A typical case for the latter is a concrete structure in marine environment, but there are other important cases where this can happen, most notably in concrete bridge decks as well as garage and industrial floors (Popovics et al. 1983).

66. Sometimes the first evidence of distress is brown staining of the concrete around the embedded steel. This brown staining, resulting from corrosion (rusting) of the steel, may permeate to the concrete surface with no cracking of the concrete, but usually it accompanies cracking, or cracking of the concrete may occur shortly thereafter (Idorn 1967). Concrete cracking occurs because the corrosion product of steel, an iron oxide or "rust," has a volume much more than the metallic iron from which it was formed. The forces generated by this expansive process can far exceed the tensile strength of the concrete with resulting cracking. Steel corrosion not only causes distress because of straining, cracking, and ravelling of the concrete (Figure 8) but may also cause structural failure resulting from the reduced cross section and hence a reduced bearing capacity of the steel. This is normally more critical with thin prestressing steel tendons than with large reinforcing bars. The rate and extent of cracking caused by corrosion of the reinforcement depend

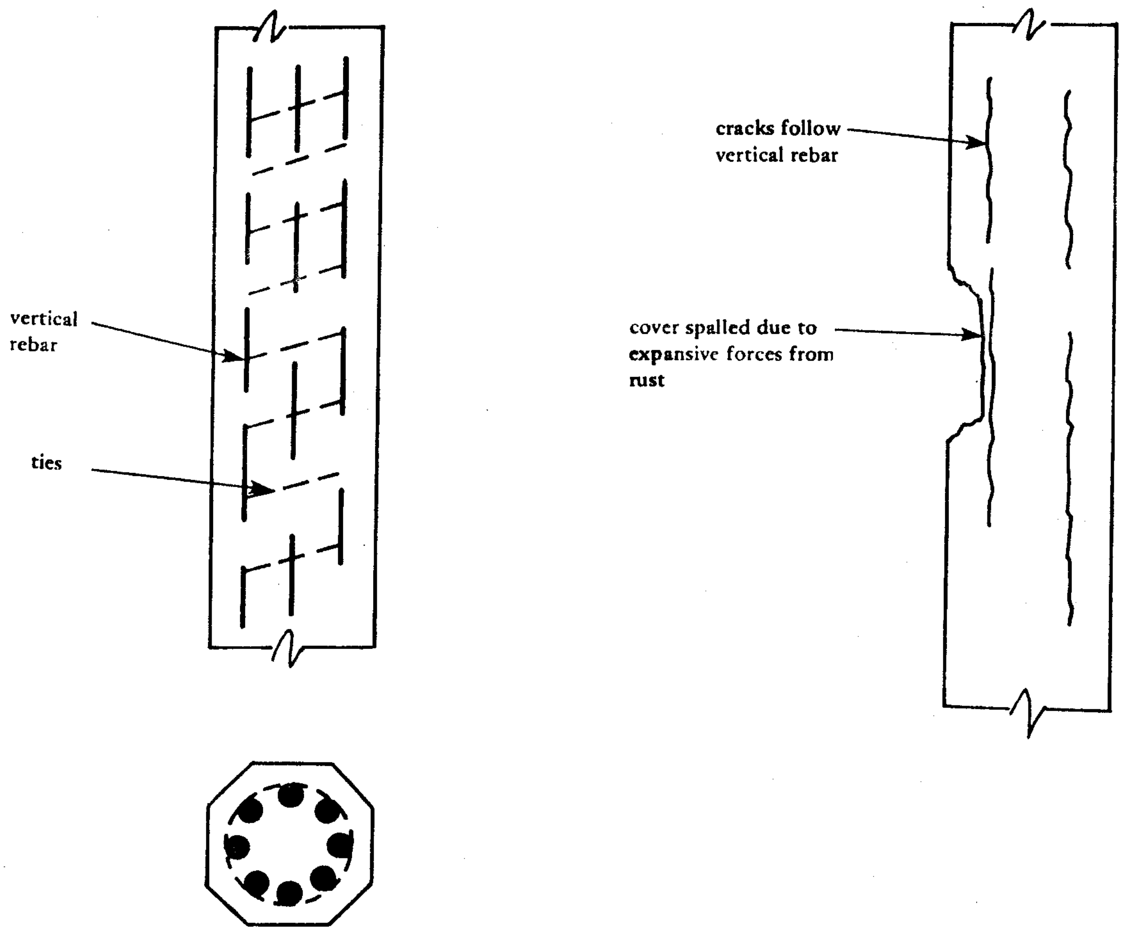

Figure 8. Examples of damage resulting from corrosion of reinforcement (from Brackett, Nordell, and Rail 1982)

partly on the properties (tensile strength, modulus of elasticity, thickness of cover) of hardened concretes and partly on the extent of rust (black or red, quantity) of the steel. This latter is influenced again by the concrete properties (type of cement, thickness of cover, permeability, diffusivity, presence of excessive porosity and/or cracks) as well as on the availability of certain ions, primarily oxygen and chlorides ions, including the characteristics of the environment. Presently, it is not possible to evaluate the effect of each of these factors separately on the corrosion of reinforcement and the resulting cracking of the concrete.

67. A frequently used method for the reduction of corrosion of reinforcement is the coating of the steel bar with a suitable, noncorrosive material. Zinc and other metal coatings have been used with certain success for a long time (Zinc Institute 1981). Recently nonmetallic (epoxy) coatings have

been introduced and have shown good results (Clifton, Beeghly, and Mathey 1975) although even these cannot ascertain perfect protection against corrosion.

68. Some of the principles concerning the durability of concrete structures in marine environment are as follows (Hansen 1965, RILEM-AIPCN 1965):

- *a*. Rusting accompanied by expansion of reinforcing steel is the greatest cause of deterioration in concrete structures in seawater. Concrete deterioration is usually significant in itself in that it may destroy or weaken the concrete surrounding the reinforcing steel.
- *b*. The most important environmental factor concerning the durability of concrete structures in seawater is the position of the concrete in relation to seawater level. Concrete between tide levels is more susceptible to deterioration due to frost action, to wetting and drying, to the capillary rise of seawater in the pores of the concrete, and to attrition from wave action.
- *c*. For reinforced concrete, the degree of exposure immediately above high water level or where sea spray can reach the concrete may be even greater than between tide levels. It is in the upper portions of reinforced concrete piles and on the underside of decks, or beams supporting them, that corrosion is most intense.
- *d*. A fundamental factor in the resistance of concrete to deterioration due to seawater is for it to be penetrated as little as possible by the seawater.

Attacks by Waters Other than Brine

69. Sulfate solutions, predominantly sodium and magnesium, are among the most frequently occurring groups of aggressive substances in aqueous solutions. Such sulfate solutions occur in industrial wastewaters as well as in the natural waters because soluble sulfates are leached from or are part of the soil solution of many soils, especially clayey soils. These sulfate solutions can cause spectacular damages in the concrete within a few years. Sulfate attack was discovered and identified as early as the beginning of the century. For instance, on the basis of chemical analysis (Burke and Pinckney (1910)), sulfates react with the calcium hydroxide produced by the hydration of portland cement; the resulting new compounds may have good cementing properties but occupy more space than calcium hydroxide. This increase in volume

disrupts the hardened cement paste, causing it to bulge, crack, and crumble. In other words, the role of calcium hydroxide in the sulfate attack was correctly recognized. Later, the important role of the aluminate compounds in portland cement was also recognized. Thus, it is known that the deterioration caused by the sulfate attack is chemical in nature because it is caused by the gradual formation of crystals of ettringite which is a calcium aluminate sulfate, and contains about 30 molecules of water. Due to the large quantity of bound water, the volume of ettringite is much larger than the sum of the volumes of calcium hydroxide and C_3A components. Therefore the internal pressure generated by the formation of ettringite is quite large, causing first volume increase, and later cracking and crumbling of the concrete. An accompanying chemical phenomenon is the combination of sulfate with the liberated calcium hydroxide in the cement paste to form gypsum which also has a larger volume than the calcium hydroxide (Swenson 1968). In addition, a purely physical action, the crystallization of the sulfate salts in the pores of the concrete, can cause considerable damage and is a Class V deterioration. This can be especially severe when evaporation takes place from an exposed concrete face or in the case of intermittent immersion to a sulfate solution. It has also been observed that a magnesium sulfate solution is more aggressive than a sodium sulfate solution of the same concentration because the magnesium cation itself has a contributing deteriorating effect.

70. Copper sulfate, manganese sulfate, and ammonium sulfate are also aggressive substances. It has also been documented that concrete specimens under sustained uniaxial tension exhibit highly reduced sulfate resistance as compared to that of comparable but unloaded specimens (Simeonov and Nazurski 1979).

71. A recommendation of ACI Committee 201 (1977) offers a tabulated form of some of the protective measures against sulfate attack. The ACI values also apply to areas in the splash or spray zone.

72. The values are applicable to structural lightweight concretes except that the maximum water-cement ratios of 0.50 and 0.45 should be replaced by specified 28-day strengths of 3,750 and 4,250 psi, respectively. It should be noted that Type V portland cement has less resistance against the attacks of magnesium salts, such as magnesium sulfate, than sodium sulfate.

73. Since different portland cements can have varying sulfate resistances, especially when pozzolanic materials are also used, a test method is

needed for the prediction of sulfate resistance of the concrete in question. The best assessment of sulfate resistance of a given concrete is obtained by long time exposure tests. CRD-C-211-84 (USAEWES 1949) provides a standard test method for analysis of some types of sulfate resistance.

74. Sulfate attack, and in general, deterioration caused by other factors, can also be investigated by determining changes in the weight of the specimen, depth of the deteriorated concrete layer, appearance, strength, the diffusion coefficient, or by determining the chemical changes in various depths of the concrete or mortar specimens after exposure to a sulfate solution. Sonic methods can also be used for this purpose.

75. The cement paste is chemically basic, having a pH of about 13, therefore is attacked by water of low pH, containing any organic or inorganic acid. The reaction is typically the dissolution of both hydrated and unhydrated cement compounds to form water soluble reaction products resulting in Class IV deterioration. Blended cements have no better acid resistance than a pure portland cement. Neither does the type of the portland cement make any difference. Limestone and dolomite aggregates are also susceptible to acid attack and may cause concrete deterioration, but in other instances they may function as sacrificial materials using up the acid, thus prolonging the service life of concrete. The rate of deterioration depends not only on the rate of the chemical reactions but also on the ability of the acid to penetrate into the hardened concrete and on how quickly the reaction products are leached or removed from the concrete.

76. As a rule, stronger acids produce deteriorations more quickly. An acidity in a pH range of 5.5 to 6 may be considered the practical limit of tolerance of high quality concrete in contact with any acid (Woods 1968). For stronger acids concrete needs protection usually in the form of a protective barrier system, the extent of which depends on the service conditions.

77. Nearly pure waters or soft water (snow water, rain water, etc.) can have high dissolving capability. Thus, they dissolve calcium compounds from hardened cement paste causing Class I deterioration. Although the capacity for dissolving lime is increased when the water flows around the concrete at a high velocity, under high pressure, or when the water contains carbonic acid, the extent of deterioration is rarely serious with concrete of good quality. Nevertheless, conventional portland cement concrete without extra protection

is not recommended for conducting hot, mineral-free distilled water (Tuthill 1978).

78. Marsh waters may have greatly differing compositions ranging from nearly pure to aggressive solutions. Typical examples for this are when carbonic acid, humic acid, sulfate, or sulfuric acid is present, singly or in combination.

PART III: PREINSPECTION ACTIVITIES

Background Information for the Structure

79. The planning, proper performance, and evaluation of inspection reports cannot be done without the knowledge of or the consideration of pertinent data related to the design, construction, operation, and maintenance of the structure in question. The collection of all this background information may be quite difficult because some of the information may be scattered, may not be available, or perhaps may not exist. Potential sources for background information are:

- a. Administrative offices (district, state and local, project headquarters).
- b. Companies (designer, contractor).
- c. Individuals (owner, operator, or former owner and operator, active or retired design and project engineer, project operation personnel, inspector, tradesman).
- d. Newspapers and magazines.

Note also that new testing may provide some of the needed data.

80. Examples of sources for the needed background information taken from a list by Stowe and Thornton (1984) are as follows:

- a. Geologic information. Geologic data, exploration logs, drill scores.
- b. Design information. Project description, design criteria, plans, and specifications, including foundations.
- c. Information on concrete. Quality and quantity of the materials from which concrete was made, concrete composition, mixing and delivery, concrete quality control data, materials testing records, placement of concrete, and batch size.
- d. Information on construction. Construction contract and records, climatological and environmental data.
- e. Analytical data. Assumed loading conditions, stability and strength analysis calculations, foundation studies.
- f. Operation information. Water level, ice or temperature extremes, boat, ship, or any other dynamic impact, increased structural loads or loadings, and other changes in operational procedures.
- g. Information on maintenance and repair. Dates, locations, types and extent of maintenance and repair activities, repair materials and techniques.

81. Routine and periodic inspection reports, condition survey reports, and instrumentation observation records are additional sources of background information that can be helpful in inspecting and evaluating underwater concrete in service. The Corps of Engineers publication, "Periodic Inspection and Continuing Evaluation of Civil Works" (HQUSACE 1983), has resulted in thousands of inspection reports. These can be helpful in providing background information on a particular dam of interest. The existing and/or potential hazards of various dams to life and property are cited in these reports. These reports can also serve as patterns for the documentation of conditions of other concrete structures.

Planning of Inspection

82. Once information about the facility has been collected and evaluated, an inspection plan should be developed. The complexity of inspection and evaluation of underwater structures requires careful planning of the related activities. The flow chart in Figure 9 may be helpful to define inspection data needed for proper evaluation of the condition of an underwater structure. Of critical importance to the effectiveness of each survey is the proper and adequate selection of the areas to be examined. It is important to select a sufficient number of inspection areas to provide representative information on the overall structure. Making this selection requires an understanding of the facility and structural analysis to determine which areas are subjected to maximum stress, fatigue, and impact forces. A knowledge of deterioration and damage theory is also useful. Consequently, the inspection plan must be prepared in cooperation with qualified engineers familiar with the structure. The inspection plan should include the identification of the inspection equipment most appropriate to the specific tasks (HAN-Padron and NCEL 1984).

83. Many problems are associated with improper or neglected maintenance of substructures located under water. As soon as deterioration or distress of a substructure has been detected, it is essential that the basic cause of the problem be determined. Failure to determine the cause or to understand the deterioration process may lead to an improper repair procedure. Also, if cracks, spoils, voids, or inadequate cover over reinforcement in the concrete are not repaired in time, distress or failure may occur. To implement and

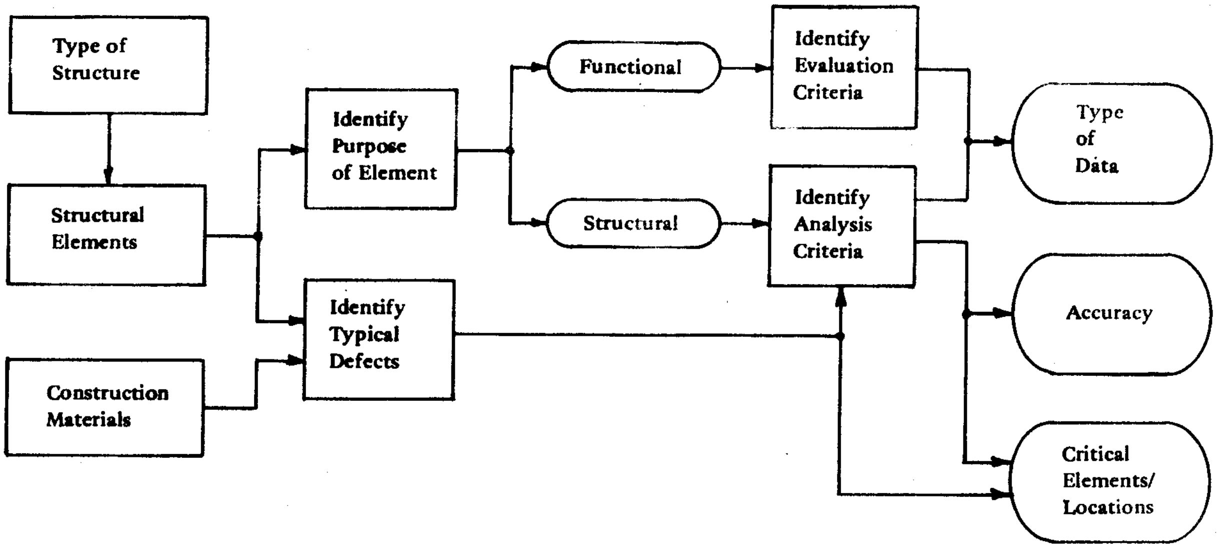

Figure 9. Procedure for defining inspection data requirements for underwater structures (from Brackett, Nordell, and Rail 1982)

assure a proper maintenance program for underwater structures, it is essential to conduct periodic underwater inspections.

84. Underwater inspections should be conducted during and at the end of the construction phase (after pile driving, underwater concrete placement, etc.), before acceptance of the structure because the discovery of any damage (cracks, spoils, honeycombs, splits, deformed or crippled areas, etc.) becomes more difficult and expensive with the passage of time. The recognized capability of an agency to perform reliable underwater inspection may prove beneficial in performing top-quality underwater construction. Also, underwater inspection after completion of the structure provides data for valid determination of compliance with the contract specifications and details, and provides an inspection baseline for future underwater inspection. Future underwater inspections should be scheduled during the period of the year when conditions are most favorable, such as during periods of low water and low pollution levels, minimum ice, or good underwater visibility.

85. The frequency of additional underwater inspection varies among agencies usually from once a year to once every 5 years. In addition, in several states bridges are inspected after each storm or collision (Lamberton et al. 1981).

86. The following list, which is based on a similar one from Stowe and

Thornton (1984), contains several steps that may be involved in planning:

a. Meet the client and define the problem (the office, or at the site is appropriate).

b. Determine the objective and scope.

c. Select the type and level of inspection.

d. Select the technical evaluation team.

e. Establish investigative procedures and techniques.

f. Establish procedure and technique for underwater data collection.

g. Select the underwater team.

h. Collect and review engineering and geological data and other background information.

i. Prepare a checklist for the data collectors concerning the information to be obtained.

j. Specify the field investigation.

k. Select the communication system to be used (notebooks, photos, video tape, etc.).

l. Specify the laboratory investigation.

m. Analyze findings.

n. Make engineering assessment.

o. Make management decisions.

p. Report findings.

87. It helps the planning if one or more working hypotheses are developed from preliminary information which are intended to explain the reasons for the condition of the concrete in question.

88. Whether or not samples of hardened concrete are needed for laboratory tests (Level III inspection) can usually be determined only after a preliminary inspection. Nevertheless, it is recommended that a sampling procedure be included in the plan. The locations as well as the size and number of specimens are determined by the prevailing condition of the structure.

89. Concrete of underwater structures is sampled usually by taking cores. A general sampling procedure is specified in CRD-C 26-83 (USAEWES 1949). Further details can be found in two pertinent reports (Stowe and Thornton 1984, Lamberton et al. 1981), especially as related to underwater sampling.

Cleaning

90. A report by Keeney (1987) states that

> "Underwater cleaning is required to remove fouling (marine growth), corrosion, and debris from the submerged portions of structures to facilitate inspection, maintenance, and repair operations. Surfaces must be free of all fouling and debris to allow a thorough visual examination and accurate condition assessment of the structure. Cleaning is also required before most forms of nondestructive evaluation can be conducted."

This report further lists several factors which can determine the effectiveness of the cleaning procedure used for a particular application as follows:

- a. Physical and operational characteristics of the cleaning device.
- b. Amount and type of fouling.
- c. Construction material.
- d. Operator experience.
- e. Underwater working conditions.
- f. Surface accessibility.

91. According to Lamberton et al. (1981):

> "Indiscriminate cleaning should be avoided. Cleaning is not only arduous but can also consume a great amount of time. Another reason to avoid unnecessary cleaning of marine growth is that the growth can afford some protection against certain types of deterioration.
>
> A wide variety of tools are available for cleaning marine growth under water. Light cleaning is usually performed with a diver's knife or hand tools such as chipping hammers or scrapers. The cleaning of thick, hard growth from large areas is best accomplished with power tools, such as pneumatic or hydraulic-powered chippers, grinders, and brushes. Sand or water blasters can also be used under water.
>
> For tough cleaning jobs, a high-pressure water blaster is effective for steel and concrete surfaces. Water blasters with discharge pressures up to 15,000 psi are available. The system consists of a surface pump, a high-pressure hose, and a gun. The gun nozzles vary in orifice size and type; usually a fan jet nozzle can be used on concrete. The gun control is a pistol grip, and the gun flow is divided into a primary flow for cleaning and a secondary flow to counter-balance the jet. These high-pressure models are capable of cutting

through sound concrete; thus care must be taken not to damage the structure. The water blaster is cumbersome and potentially dangerous; the diver must be careful to keep the nozzle end pointed away from himself and others as it will remove bone and muscle more easily than fouling organisms. It should, therefore, be used with great care and only by an experienced operator. It should not be not be used on timber structures.

The hydraulic-power brush systems used to clean ship hulls can also be used to clean large concrete surfaces under water. These systems are not as rapid as water blasters but are satisfactory for some cleaning tasks. Hull scrubbers are similar in operation to floor buffers. The centrifugal action of the brush causes it to adhere to the surface being cleaned. By lifting one edge or the other, the brush can be made to move in any direction; the diver simply hangs on and goes for a ride. These brush systems are available in various sizes; brush sizes up to 16 in. diameter are suitable for concrete cleaning and can be operated out of small (18 to 20 ft long) dive boats."

PART IV: UNDERWATER INSPECTION

Nature of Inspection

92. RILEM Recommendation TBS-1 (RILEM-AIPCN 1965) for underwater inspection of concrete structures states that the nature and technique of the observation is completely dependent on the type of work to be studied and on the structural characteristics that influence its safety. Factors to be considered in observation of concrete structures, as well as the equipment that can be used to quantify these observations, are presented below:

a. Cracks:

(1) Whenever a crack is detected, it should be marked by a waterproof line along the crack at a distance of 0.1 to 0.2 in. The extreme points, the beginning and the end of the crack, should be denoted by orthogonal strokes, and the dates of observation should be indicated at these points.

(2) In zones where the crack is wider, the reading point should be marked by an orthogonal stroke and by the reading reference. Readings should be carried out by using a suitable magnifying lens and scale for measuring crack widths.

(3) If future observations show that cracking has progressed, both the development of the earlier crack and the appearance of new cracks should be signalled and their widths should be measured. Cracked zones should always be surveyed, and the corresponding drawings must be clear to indicate the location of these zones in the structure. Photographs of the most representative zones should be obtained.

b. Foundation problems:

(1) In general, the foundation of a structure undergoes settlements without causing danger. If, however, those settlements are of differential nature, severe deficiencies may result in the structural behavior. Whenever a differential settlement of the foundation of a structure occurs, geometric leveling must be carried out with suitable topographic equipment or with a liquid leveling system.

(2) Scour is another potential problem that should be checked. This is the removal of stream bed, backfill, slopes, riverbed rock, or other supporting material by moving water.

c. Joints:

(1) Joints are delicate bodies in any structure since they always introduce a discontinuity in structural elements. Particular attention should be paid to them whenever their movement clearly exceeds what was anticipated in the design.

(2) Whenever there is ground for suspicion of any kind of anomalous operation, the relative displacement of the two edges of the faulty expansion joints should be measured. For measurements, vernier calipers or any other device providing readings with errors up to 0.1 in. on reference bases clearly identified on the joints are recommended.

d. Permanent deformations: Planes of linear structural elements may present permanent deformations noticeable on sight. Whenever there is reason to suspect that there is some anomalous permanent deformation, it should be marked off and measured with the means available onsite, and values quantifying the deformation should be recorded.

e. Structural stability: Proper functional behavior of the anchors is of basic importance for the overall safety of the structures. Therefore, they should be observed very carefully to detect any possible fissures in the enveloping materials in the base or sliding or in the end parts of the cable which convey large loads. When there is reason to suspect that cables or ties may slide on their anchoring parts, specific instructions should be given to check these slidings by measurements.

f. Alignment and verticality:

(1) Linear displacements in the vertical or horizontal directions can be checked with simple methods. These can also be used to check the verticality of walls, columns, or any other upright parts in a structure.

(2) The checking of alignments under water can be carried out essentially in the same way as above water, that is by unaided eye or by using a fine steel wire duly stretched between reference parts, which is a simple but accurate device for most cases.

(3) For measuring deviations from the vertical, one should always mark fixed references in the structure and improvise plumblines with steel wires or wires of other strong material suspended from a reference point that should always be in the same position. The measurement of the distance between the wire and the other references is carried out by using a scale.

g. Erosion:

(1) Erosion is a frequently occurring damage in underwater structures. It usually appears in the form of a worn surface, a cavity, or both; therefore, it is noticeable on

sight or, in turbid water, by tactile methods. The visual underwater inspection can be done from above the water under certain circumstances, for instance, by an underwater scope.

(2) The size of the erosion should be measured by the means available onsite. Furthermore, the nature of the erosion, the sand, rock, concrete pieces, and other debris present, as well as the velocity and other characteristics of the water flow should be described. This description should be supplemented by photography or video tape whenever possible.

h. Corrosion of reinforcement: Although corrosion is easily noticeable due to the iron salt stains which may be observed on the walls of structures, particular care should be given to detecting corrosion of reinforcements, for instance by means of hammer blows on the surfaces of the concrete under observation. In case of corrosion, the sound of the hammer blow becomes dull as it indicates the presence of a significant amount of rust.

Typical Inspection Procedure

93. Typical procedures for inspection of underwater concrete structures are as follows (HAN-Padron and NCEL 1984):

a. Inspect the structure beginning in the splash/tidal zone (Figure 1). This is where most mechanical, physical, and chemical damage is normally found.

b. Clear a section about 18 to 24 in. in length of all marine growth. Visually inspect this area for cracks with rust bleed, broken pieces caused by spalling of mechanical damage, and exposed reinforcing steel.

c. Sound the cleaned area with a hammer to detect any loose layers of concrete or hollow spots in the pile or structure.

d. Descend, visually inspecting the pile or structure where marine growth is minimal, and sound with a hammer.

e. At the bottom, record the water depth along with any observations of damage, for instance on a Plexiglas slate.

f. Upon return to the surface, immediately record all information into the inspection log.

g. Topside personnel observe all features of the structure above the water line and record any damage found.

h. Inspect in greater detail the base of mass concrete structures, such as retaining walls and foundations. These types of structures are prone to undermining by wave and current action which, if not rectified, could lead to failure of the structure.

94. A more detailed inspection procedure is described in Appendix A for underwater seals, footings, and piles.

Inspection Techniques for Concrete

95. Various methods are presently in use or currently under development for the evaluation of concrete under water. A summary of these test methods is presented by Lamberton et al. 1981, Rissell et al. 1982, Brackett, Nordell, and Rail 1982.

96. All of these techniques require the diver to locate the area of the defect, make the measurements, and record the results or relay them to the surface by means of a diver communications system. In every case the diver must be knowledgeable about the "normal" condition of the structure and types of damage/defects which may be encountered.

97. Experience requirements for the diver may be reduced in many instances by using an underwater video system. This allows both real time observation of the inspection by a facilities engineer and video tape documentation of severely damaged areas.

Visual inspection

98. Visual inspection is invariably the first test that is applied for underwater evaluation. Its purpose is to confirm the as-built condition of the structure and detect severe damage to the structural elements (Level I inspection), and to detect surface damage (Level II inspection). In certain cases this can be done from above the water by using an underwater scope. For instance, in the case of the repair of erosion in the Chief Joseph Dam, engineers made underwater observations through the use of a 35-ft-long underwater scope equipped with a 6-in.-diam bottom glass and a high-power telescope (McDonald 1980). Visual inspection is quick, easy, and nondestructive. However, there are numerous limitations to this type of inspection technique resulting from the environment, the diver, and defects which are not obvious on the surface of the structure. Environmental limitations include: (a) heavy fouling by marine organisms which obscure surface defects unless cleaned; (b) poor visibility; and (c) strong currents, which make it difficult for the diver to work. Diver-imposed limitations range from inadequate training as an inspector to reduced attention resulting from cold or reduced visibility in water. For instance, when turbidity is high, underwater visibility can be nonexistent, even if the diver carries artificial lights, as direct light is absorbed by turbidity. However, the placement of the lights at 45 deg to the axis of the camera reduces the reflectance of the light which increases

visibility. The wavelength of the light affects penetration in highly turbid water. There has been substantial investigation in the industry of the use of quartz iodide and thalium iodide lamps to improve visibility. Since the intensity of the lights affects visibility, in this case the addition of more lights is not the better solution. The objective should be to expand the lighting on the entire area. Visibility may also be improved if a clear-water mask is attached to the face of the diving gear. Small plastic bags (0.5 ft^3) can be used as a clear-water lens.

99. Additionally, visual inspection is only qualitative in nature and does not provide information about concrete strength.

100. Tools for probing, scraping, measuring, and recording underwater findings are necessary for a visual inspection; hammers, picks, pry bars, probing rods, and similar devices should be available, along with diver's lights, spare batteries, and still-camera equipment. A diver's slate is useful for making notes and sketches under water.

101. Although visual inspection can reveal many defects, it cannot reveal all. This should be considered when planning an inspection and judging the condition of a structure based on visual data.

Tactile

102. Since the substructures of many structures are in turbid water that severely limits visibility, the diver must touch and feel to detect flaws, damage, or deterioration. Divers are capable of conducting inspection using only tactile methods in zero visibility, yet it is difficult, if not impossible, to assess the value of this technique. The task is even more difficult in cold water when there is a strong current, or when the structure is coated with marine deposits.

103. Tactile inspection requires greater preparation than when working in clear water. Attention should be given to the following items:

a. In-depth study of plans.

b. Good communications between diver and surface.

c. Blind practice in a pool to help assess capability.

d. Use of nondestructive testing and other techniques not dependent on sight or touch.

e. Increased use of sounding equipment to detect scour, debris buildup, etc.

f. Recorder to document voice transmission.

g. Determination of position and depth of diver.

Physical dimensions measurements

104. The measurement of physical dimensions provides direct information about section loss of concrete and reinforcing steel. This method is quick and easy, results are quantitative, and it is economical even under water because of the minimal amount of time and equipment involved. The method is nondestructive, applicable to underwater use, and provides a partial measure of the member's strength. Its primary drawback is that it does not provide information on the strength of the remaining concrete which is generally in question at this well-advanced stage of deterioration.

Acoustic ringing

105. Acoustic ringing is done by striking the concrete surface with a hammer to locate areas of loose internal structure, delamination of the concrete cover caused by the effect of freezing and thawing, and/or corrosion of the reinforcement. The method is economical but is not particularly effective because it is qualitative only in nature, and the inspector's ability to hear sound in water is reduced by waves, currents, and background noise.

Cores

106. Cores are taken under water to provide specimens for further tests of the concrete. Cores permit petrographic analysis and other laboratory procedures that measure compressive strength, diffusion constant, permeability, electrical resistivity, density, X-ray diffraction, moisture content, chloride penetration, extent of carbonation, and entrained as well as entrapped air contents.

107. The technique requires a coring drill equipped for underwater use and a materials laboratory. The method is destructive in that it causes microcracks and leaves holes. Coring gives a direct measure of strength, but because it is moderately expensive, cores are typically taken only when other evidence indicates that further investigation is warranted.

Pulse velocity

108. Pulse velocity is determined by measuring under water the time of transmission of a pulse of energy, usually ultrasonic, through a known distance of concrete. The velocity of the pulse is proportional to the dynamic modulus of elasticity which, in turn, infers concrete strength. The results can be affected by many factors including aggregate content and reinforcing steel location. The results obtained are quantitative, but they are relative

only in nature. Pulse velocities need to be correlated with other tests such as corings, in order to obtain absolute values. That is, the test is not reliable enough in its nonrelative form for direct strength determination but evaluates homogeneity and crack location quite well.

109. A special form of this technique is the pulse-echo method. The application of this is illustrated in Appendix B for the in situ determination of the length and condition of concrete piles.

Other sonic methods

110. Echosounders (specifically fathometers) are effective in checking scours in the streambed. The exception is when this method is used very near a vertical structure, and erroneous returns from the structural elements under water may occur. This is usually not the case for horizontal or inclined structures. Undermining of piers or abutments cannot be adequately detected with an echosounder. Therefore when undercutting or undermining is suspected, there is no substitute for thorough visual observations.

111. A side-scan sonar system is similar to the standard bottom-looking echosounder except that the signal from the transducer is directed laterally and produces two side-looking beams. The system consists of a pair of transducers mounted in an underwater housing or "fish" and a dual channel recorder connected to the "fish" by a conductive cable. The recorder initiates the signal to be transmitted by the transducers and after a period of time, depending on the distance the signal travels, the reflected return signal is received and appears as a darkened area on the chart recorder. The more reflective the object illuminated by the signal is, the darker the object appears on the record. Directly behind this return appears a shadow area, the size of which is dependent upon the size of the object illuminated, the angle of the signal to the object, and the angle of the slope behind the object. The various shades of gray indicate changes in texture and relief on the bottom (Patterson and Pope 1983).

112. A major concern in the application of the side-scan sonar technique to observe underwater structures along coastal slopes is the resolution (the ability to distinguish one object from another) offered. Since the goal of the inspection work is to assess the condition, coverage, or damage, resolution is desirable. In one successful case, a 500-kHz transducer was selected for this purpose because it demonstrated the capability to resolve

substantial details of objects lying on the slope.* It is also important that the "fish" is fixed in relation to the boat or the water surface, otherwise elevations at the bottom are not attainable. Bottom elevations may be desirable to monitor degree of deterioration from known elevations.

113. Side-scan sonar has become a widely used tool in numerous ocean applications since its introduction in the late 1950's. Its unique ability to rapidly produce photograph-like images of the seafloor makes it the instrument of choice in searching for lost objects or for mapping of the bottom areas.

114. In the past several years the side-scan technique has been used to map surfaces other than the ocean bottom. Successful trials have been conducted on the slopes of ice islands and breakwaters and on vertical pier structures (Mazel 1984).

115. The results and conclusions to the side-scan sonar survey of underwater structures indicated that this technology is both possible and practical in documenting construction as well as change occurring to the structure being studied. The side-scan sonar technique permits a valuable broad-scale view of the underwater structure that has not been obtained by any other means previously used. The results are proven to be repeatable, and sequential surveys can be used as a tool for monitoring the condition of the structure throughout its service life by a comparison of the records from surveys. Although individual details are not always discernible on the sonograph, the resolution is usually sufficient to establish the pattern and to detect irregularities within that pattern.

116. Further applications of side-scan sonar to underwater engineering are: quality control during and after construction, site reconnaissance to define existing structure and site features, and monitoring change to existing structures by periodic comparison to determine failure or observe major changes. The side-scan sonar technology has progressed to offer the engineering community a valuable tool with which to assess the quality and performance of underwater structures. With continued research this tool can become an integral part of the evaluation of many underwater projects.

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* One of the commercially available models is the Klein Associates Model 422S-101EF, 500-kHz sonar. A Klein Model 612 Alphanumeric Annotator/Digital Expansion Unit is also available to annotate the field records and to expand the data.

117. The next step would be to conduct trials on a test section of a concrete structure in which various flaws had been induced artificially. This would show how the sonar should be oriented with respect to the structure and just what it was capable of seeing.

118. Another pertinent method is called acoustic mapping. It is suitable for the performance of rapid, accurate survey of submerged horizontal surfaces. The method is described in Appendix C.

Voltage potential readings

119. Voltage potential readings are taken to assess the state of corrosion of the reinforcement by making an electrical connection to the reinforcing steel. Once the connection has been made, the test is easily performed and is nondestructive. A porous tip electrode connected to a high impedance voltmeter is placed directly on the concrete surface over the reinforcing steel and a comparison is then made of the potential between the steel and a standard reference electrode, usually silver-silver nitrate underwater and copper-copper sulfate in the splash zone.

120. The most satisfactory results are obtained when the readings are used in conjunction with the chloride penetration analysis of a test core. The readings indicate whether or not corrosion is active and the extent of the concrete surface area involved. However, it gives neither the rate nor the amount of corrosion, and no indication of strength is provided.

Computer-assisted tomography

121. Computer-assisted tomography scanning uses a nuclear source to develop a cross-sectional view of a member. It yields information on the size and location of aggregate, cracks, and voids; density; size and location of reinforcing steel; and extent of corrosion. This method is nondestructive and can scan members up to 3 ft thick. The method is very expensive, gives no direct measure of strength, and poses a potential health risk to the user. Its underwater application is presently experimental and is under development in the United Kingdom and at the University of Texas.

Polarization-resistant measurements

122. Polarization-resistant measurements can actually measure the rate of corrosion. Several techniques are possible to apply this technology. The best candidates are the AC impedance (refers to flow opposition in electrical current), the two-electrode, and the three-electrode methods. The AC impedance method has not been researched extensively because of potential equipment

problems in field use. The two-electrode method applies a 20-mV potential between the electrodes, with one electrode as the rebar being measured. The current necessary to produce the change is noted, and the polarity is then reversed to allow determination of the average current. This method has several limitations, however.

123. The three-electrode method is the most reliable. One electrode is the rebar with the corrosion rate being measured. The second electrode serves as a source of current during polarization, and the third electrode is a standard reference on which measurements are made. The normal potential of the steel is modified by an external current similar to the two-electrode method. The resulting voltage and current give an indication of the rate of corrosion.

124. Various techniques to analyze the resulting data include: slope of the polarization curve near zero current method, three-point method, graphic analysis, and "break in the curve" method. The test is nondestructive except for an electrical connection to the reinforcing steel. This technique has not been developed for underwater testing thus far and is still under research for field testing in-the-dry.

Radar techniques

125. Radar penetrates paving materials. It is refracted by any discontinuity in the material, such as boundaries between water and concrete. Changes in shape of the refracted wave form indicate changes in material, or the occurrence of some discontinuity. Thus, concrete inspecting radar (CIR) can be used to evaluate the condition of concrete up to 30 in. in depth; that is, it can differentiate between serviceable and deteriorated concrete. The deterioration can be delaminations, microcracks, and cracks up to and including small voids. CIR can also detect changes in material and locate where these changes occur (Alongi, Cantor, and Alongi 1982). When a concrete is slightly less than good, but not appreciably deteriorated, radar comparisons show an uncertain transition zone.

126. This ability of CIR to identify the condition of concrete has been established above water. For instance, CIR was used to detect voids in pavements in the 1970's (Lundien 1972). Field studies have shown excellent reproducibility (Cantor and Kneeter 1982, Cantor 1984).

127. Other laboratory and field experiments showed the possibility of identification and detection of buried containers by radar (Lord, Koerner, and Freestone 1982; Bowders, Koerner, and Lord 1982). Radar techniques may be

applicable, in principle, for underwater inspection; however, further research is needed before field application for inspection purpose.

Laser mapping

128. Airborne laser systems have demonstrated enormous potential for topographic and bathymetric mapping. Both profiling and scanning systems have been evaluated for terrain elevation mapping, stream valley cross-sectional determination, and nearshore bottom profiling. Performance of the laser systems has been impressive and for some applications is comparable to current operational accuracy requirements.

129. The general advantages of airborne laser mapping include the ability to:

a. Generate data sets in seconds which would require days or weeks by ground survey team.

b. Acquire data densities in orders of magnitude greater than those feasible for ground systems.

c. Acquire data in areas inaccessible to ground survey crews.

d. Collect data in a digital form that leads to easy and immediate computer processing.

130. The ability to accomplish terrain bathymetric mapping (in reasonably clear water) has been demonstrated. Perhaps the most serious constraint to improving the performance of airborne laser mapping systems is the adaption of improved positioning technology. Improvements in laser systems can enhance current capabilities but to a lesser degree than improvements in positioning (Link, Krabill, and Swift 1982).

131. Like other remote sensing tools, airborne laser mapping systems are certainly not the ultimate answer for all surveying and mapping problems, but, when used in areas where applicable, these systems are extremely cost-effective.

Other methods

132. The literature also revealed other methods that may be used for concrete strength determination but have not yet been developed for underwater use. These methods conceivably may be developed and used in the future. The tests are:

a. Pull-out test.

b. Penetration test.

c. Indentation test.

d. Resonance test.

133. At present, most of the states use cores, sounding, physical dimensions, visual assessments, and engineering judgment to arrive at values for reduced allowable stresses and cross-sectional area. These tests are likely to reveal trouble only after the deterioration process is well under way and substantial damage has already occurred. The best results are most likely to be obtained from a combination of several different tests involving a large number of samples. A wide variation in results can be expected due to the nonhomogeneous nature of concrete.

134. As a rule, destructive testing should be avoided. Any test that induces cracks or reduces concrete cover can only promote deterioration. However, if after testing, all holes are filled and all cracks are sealed, the deleterious effects of testing can be minimized. This care should be applied to all materials (Rissel et al. 1982).

135. A summary of nondestructive methods for underwater testing of concrete is presented in Table 2.

Documentation of Inspection

136. For the results of the inspection to be useful, they must be documented in a clear and concise manner, preferably in permanent form and in accordance with generally understood terminologies. Inspection forms and reports should be completed during the inspection, or as soon as possible after the inspection is completed. Standard forms and report formats greatly facilitate the documentation procedure and are essential for comparing the results of the present inspection with past and future inspections. Table 3 provides the explanation of the condition ratings for concrete piles. Recording of the observations while under water is possible with a grease pencil on a Plexiglas slate. Note however, that apart from the limited accuracy of such recording, sensory data are not always reliable because of the adverse ambient conditions under water as previously discussed.

137. Therefore whenever appropriate, visual inspection should be documented with still photography (HAN-Padron and NCEL 1984), for instance.

138. Although obtaining clear photographs in turbid water may require special equipment (Appendix D), this method is inexpensive, readily available, and simple in that very little training is required for its use. A description is presented in Appendix D.

Table 2

Sensors That Can be Used to Collect Data Required for Underwater Inspection of Structures (after Lamberton et al. 1981)

Applicable Sensors	General Structural Integrity	Structural Deformation	Joint Separation*, **, +	Joint Cracking*, **, +	Corrosion Protection System Integrity	Corrosion Potential Measurements	Corrosion Thickness Measurement*, **, +	Fouling*, **, +	Scour	Nature and Location of Debris	Vibration Signature	Tilt
Eye	X	X	X	X	X		X	X	X	X		X
Video camera	X	X	X	X	X		X	X	X	X		
Film camera	X	X	X	X	X		X	X	X	X		
Optical scan	X	X	X		X			X	X	X		
Acoustic scan	X	X							X	X		
Radiographic				X								
Profile gage		X					X					
Straight edge		X					X					
Accelerometer											X	
Ultrasonic flaw detector		X	X									
Platform tilt and level gage												X
* Brush cleaning required			X	X			X	X				
** Chipper cleaning required			X	X			X	X				
+ Water jet cleaning required			X	X			X	X				

Table 3

Explanation of Pile Condition Ratings for Concrete Piles

(after HAN-Padron 1984)

Concrete Condition Rating	Explanation
NI	Not inspected, inaccessible or passed by
ND	No defects: no cracks or fine cracks good original surface, hard material, sound
MN	Minor defects: good original section minor cracks or pits surface spalling that exposes coarse aggregate small chips or popouts due to impact slight rust stains no exposed rebar hard material, sound minor honeycombs
MD	Moderate defects: spalling of concrete minor corrosion of exposed rebar rust stains along rebar with or without visible cracking softening of concrete due to chemical attack surface disintegration to 1 in. due to weathering or abrasion reinforcing steel ties exposed popouts or impact damage moderate honeycombs
MJ	Major defects: loss of concrete (10-15 percent) one or two rebars badly corroded one or two ties badly corroded large spalls 6 in. or more in width or length deep, wide cracks along rebar dummy areas full width of face major honeycombs
SV	Severe defects: two or three rebars completely corroded no remaining structural strength significant deformation severe honeycombs

139. The photographs can be in both monochrome and color versions. The color system can provide more information, although water, especially when turbid, will filter out most colors. Monochromatic marine growth which may cover the surface of most marine structures in many cases will make photographic color distinction difficult. This problem can be reduced by preparing the structure prior to inspection by scraping away the growth or by using proper lighting. The disadvantage of delays in processing color photographs can be rectified somewhat by using Polaroid 35 mm film with a portable processor. This equipment is relatively inexpensive but lacks the same degree of photograph resolution attainable using standard 35 mm color film. Black and white photographs can be routinely processed immediately on the site.

140. All photographs should be numbered and labeled with a brief description of the subject. A slate or other designation indicating the subject should appear in the photograph. When color photography is used, a color chart should be attached to the slate to indicate color distortions.

141. The current trend is for both divers and ROV's to use video imaging systems to transmit visual data to topside personnel who are able to better analyze and interpret the video information as well as to allow for the permanent recording of the data. Cameras have been designed to be hand-held or mounted on the diver's headgear. Vocal observations can also be transmitted and recorded.

142. The <u>vidicon</u> is a tube-type device which is probably the most widely used image sensor for underwater investigations. The standard vidicon very nearly matches the human eye for spectral response and resolution. It is, however, sensitive to damage from bright light and lacks the sensitivity to low light levels. Modifications of the standard vidicon have improved the performance in these areas. Solid state image sensors are available on the market. The imaging systems also come in monochrome and color versions, the latter being the preferred version.

143. In spite of the problems relative to color pictures, imaging systems do offer the advantage of real-time display to the surface and real-time quality control of the video image. As a documentation tool, these systems offer no particular advantage over still or moving photography. As a matter of fact, it is common practice to supplement video image with still photographs (Lamberton et al. 1981).

144. Video tapes should be provided with a title and lead-in describing what is on the tape. The description should include the nature and size of the structure being inspected and any other pertinent information (HAN-Padron and NCEL 1984).

145. A debriefing with the activity personnel with slides, photographs, or tapes should be conducted before leaving the site, and all questions should be resolved.

PART V: POSTINSPECTION ACTIVITIES

Condition Assessment Using Inspection Data

146. The assessment and decision-making processes following the inspection, as recommended by Brackett, Nordell, and Rail (1982) are described in the following paragraphs.

147. An inspection is a condition survey; that is, a survey of the physical condition of an existing structure. The information obtained by an underwater inspection can be used to make an engineering assessment of the structure in terms of its capacity to carry load and its capability to be used safely for all or part of its intended purpose for a projected period of time. Thus, the engineering assessment is concerned with structural capacity, safety considerations, as well as the cause, extent, and rate of deterioration.

148. Safety of a structure is evaluated in terms of both the potential for collapse or catastrophic failure of the structure, and the likelihood of injury or death to personnel.

149. Information on the cause, extent, and rate of deterioration may be used to predict useful life of the structure and to estimate maintenance and repair action needed for the structure to remain in a safe condition and to continue to perform at the required level. This information is important for readiness planning and economic reasons, which include the allocation of resources (manpower, funding) needed to maintain a specified operational capability and level of safety.

150. Structural analysis criteria for two types of failure, along with sample calculations, are presented in the report by Brackett, Nordell, and Rail (1982).

151. When the findings of the engineering assessment are available, a management decision can be made as to what course of action to take concerning the operational use of the structure (to continue normal use, to derate, to convert to other use, etc.) and the maintenance of the structure (to repair, to replace, etc.). Such management decisions can be facilitated by quantitatively rating the condition of the structure and indicating the urgency of repair.

152. Management decisions will also require other information. Most important are the operational need for the structure (including its

importance, its use in short and long range operations plans, etc.) and the effectiveness of the structure in meeting the needs for which it is intended. Other management information needed includes economic considerations (such as capital investment, replacement costs, impact of maintenance on life-cycle costs, relative priorities to use available funds, etc.), appearance, environmental conditions, and other factors outside the scope of this report.

153. The place of underwater inspection in the overall maintenance and safety program is summarized in a flow chart in Figure 10.

154. In summary, the purpose of the inspection is to provide the necessary information to make rational management decisions on the allocation of resources including justification of maintenance budgets, requests for new construction, etc. and to make rational engineering decisions on what and when to repair, what and when to replace, etc.

Report

General

155. Stowe and Thornton (1984) make the following recommendations concerning reports on inspection: (a) a formal report shall be submitted to the agency or organization requesting the condition survey; (b) the report should clearly state the condition of the concrete in the structure and appurtenant structure; and (c) any dangerous conditions existing in the concrete structure and evidence of existing or potential problems in the site environs, embankments, foundation, or in electrical, mechanical, or hydraulic features should be reported to appropriate officials of the project immediately. The final report should also contain this information.

Contents of report

156. A description of the project should include vicinity, locality, plan-view maps, elevations, sections of the structures and foundation, and geologic maps when applicable. According to the ACI,

> "General purpose and operating requirements of the project and safety hazards and economic impacts involved in case of structural failure should be described. Significant structural design criteria upon which evaluation of the concrete was made and analyses, test methods, data and investigations pertinent to the evaluation should be described"

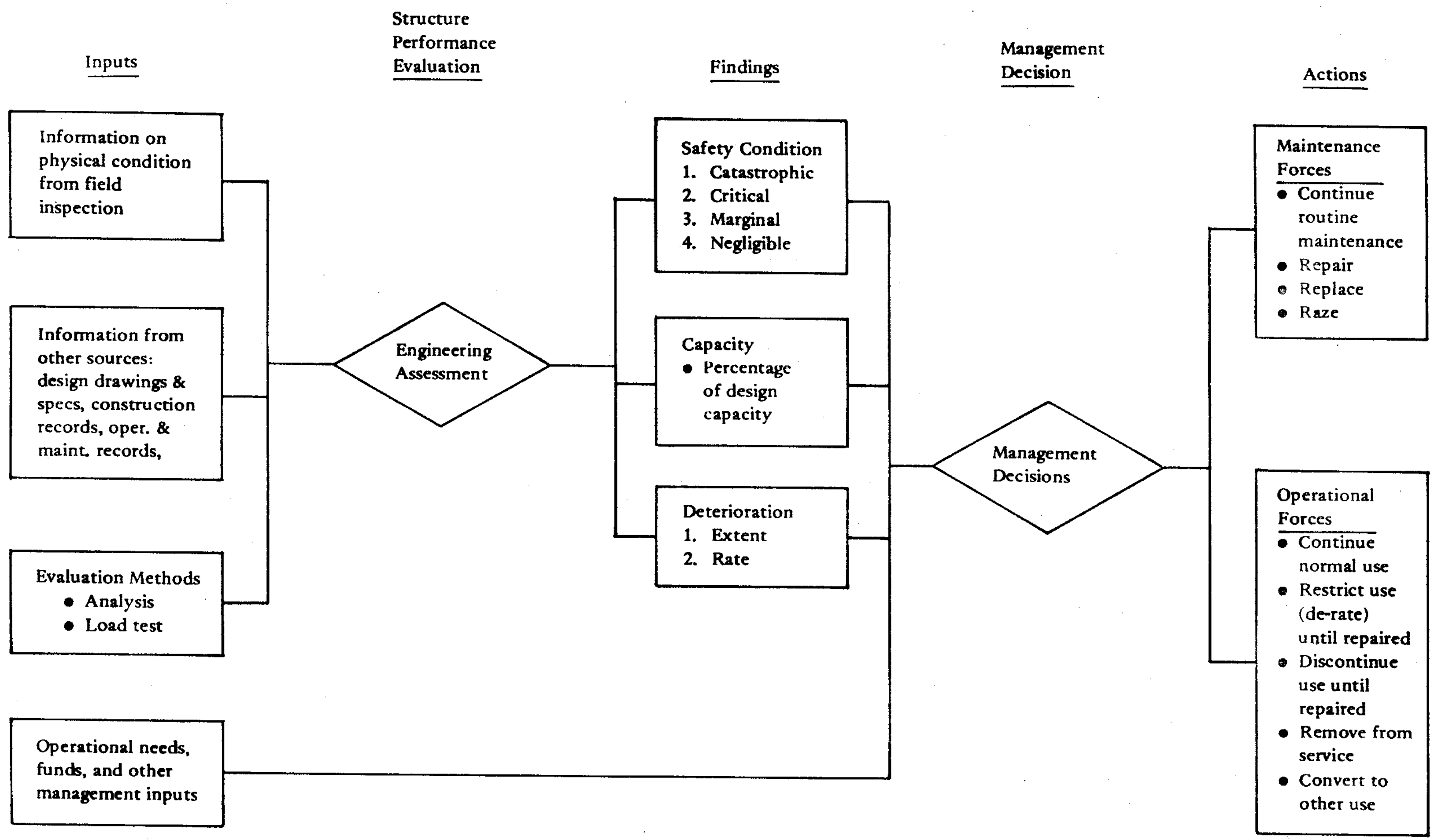

Figure 10. Decision making process for maintenance and safety program of waterfront structures (Brackett, Nordell, and Rail 1982)

(ACI Committee 207 1979b). All engineering data reviewed concerning the foundation design, construction, operations, and maintenance should be referenced.

157. The report should also contain a summary of data collection; i.e., existing records and documents; visual inspection of concrete including photographs and sketches which indicate location, extent, and depth of damaged and eroded concrete; analysis of existing instrumentation, inspections, and test records; and results and analysis of new investigations and test data.

158. The report should specify the current adequacy of concrete based on recent design criteria and service conditions. Projections of continued serviceability should also be made. When appropriate, recommendations for conventional or state-of-the-art repair should be given to assure serviceability of the structure. The Corps of Engineers has a research program entitled "Repair, Evaluation, Maintenance, and Rehabilitation" which includes an objective to develop and evaluate materials and techniques for the evaluation, maintenance, repair, and rehabilitation of civil works structures. The reader is encouraged to contact the Waterways Experiment Station for information, bulletins, and reports of work accomplished and planned on the evaluation and repair of concrete structures.

Research Needs

159. Rissel et al. (1982) list the following research needs related to underwater inspection of concrete structures:

- a. A polarization-resistant device to make underwater measurements of the actual rate of corrosion of reinforcing steel.
- b. A test block permanently attached to the substructure in a protected location to make calibration measurements. This block, which could be an outcropping on the pier cap, would aid the engineer in calibrating present and future test equipment.
- c. Provisions should be made on existing piling as well as at the time of construction of new piling for an electrical connection substructure to monitor the voltage potential for an indication of deterioration. This connection should be of trade standard quality and be situated in a protected location. Substructure units to be equipped should be in shallow as well as deep water. Determination of which units will corrode first should not be of great concern because those which do not corrode will serve as a negative control.

d. Research into the time frame of deterioration would be helpful to the maintenance engineer in predicting when a structure will need repairs. For example, if the potential for corrosion of reinforcing steel could be observed and correlated to the "time of deterioration," an approximation could be made as to when corrosion will commence and when it will become a problem. This could be accomplished with something as simple as a graph with a "family of prediction curves" on which field data points can be plotted.

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APPENDIX A: PROCEDURES FOR INSPECTING UNDERWATER SEALS, FOOTINGS, AND PILES*

General Preparation and Safety Procedures

Preparation

1. Review plans (and specs) of seals and footings, excavation sections, water conditions, and bridge and pile data.

2. Review previous inspection reports.

3. Ensure that necessary inspection equipment is working properly and is inventoried in vehicle.

Safety procedures

4. Practice basic safety procedures as instructed by certifying agency.

5. Have divers descend slowly in case of poor visibility and sharp objects along descent path or on bottom.

6. Take precautions in severe currents.

   a. Attach safety line on upstream side of bent or piles for divers' use underwater. DO NOT use safety line when visibility is poor unless line is completely taut.

   b. Stretch safety line across stream 50 yd downstream in rivers up to 300 ft wide.

   c. In tidal areas attach 100 ft of rope with ring buoy on stern of boat. Tie boat to bent or piles.

   d. Man a safety boat downstream of current.

   e. Have divers return to bent or piles to protect themselves from boat traffic.

Inspection

7. Use masonry hammer, probing rod, rule, scraper, divers' tools, caliper, increment borer, marker for a complete underwater inspection.

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* Procedures for inspecting underwater seals, footings, and piles -- a job description for divers as suggested by Lamberton et al. 1981 and adopted by the State of North Carolina (per Sandor Popovics).

Cofferdams

Inspect area at sheeting

8. A minimum of two divers will perform inspection. Each diver inspects half the perimeter of the cofferdam sheeting at base of excavation. Each diver keeps in touch with sheeting with hands or feet and moves along the cofferdam while inspecting the base of excavation from the sheeting to as far as it is possible to reach.

9. The divers should look for clay, mud, or silt buildup in all corners or between base of excavation and sheet piling throughout cofferdam. The base of each sheet piling should be examined to ensure that sheeting is driven all the way down (this is important in cases of shallow excavation depth). Check for loose, large rocks that might be leaning against sheeting.

Inspect excavation base

10. Divers will look for mud, clay, silt, loose rock, or other loose, hard foundation material. Surface should be clean of loose material. Divers will also describe geometric features and contour of surface, which can be specified as level, stepped, or serrated. Rock surfaces should be left rough.

11. Surface should be inspected for material description such as sound rock, decomposed rock, or firm clay. If surface is not sound rock, a sample of material is taken. Because specifications call for surface to be cut to firm surface, divers should be sure foundation material is what designer expected.

12. Inspect corners and all corrugations of sheeting from natural ground to base of foundation for any earth inclusions.

Inspect near center

13. When inspecting cofferdam up to 20 ft wide, at base of excavation near center, one diver keeps one hand on the sheeting while the other hand guides the other diver out near the center. The second diver inspects cofferdam while being guided and moved completely around it by the first diver.

14. When inspecting cofferdams over 20 ft wide, divers first use the method for those up to 20 ft wide and then place a rope with weight on one end along the bottom to complete inspection. This is done as one diver carries the weighted rope and stations it near center of cofferdam wall, while other diver carries other end of rope and holds on bottom at opposite wall. First diver can then proceed on each side of rope, which is used as a guideline.

Rope can be moved to a second location if inspection cannot be completed at first location.

Seals

Make layout of seal

15. Number with crayon the interior corrugations on each face of seal and record.

16. Measure distance from interior corrugation to edge of footing on all seal sides at each corner of footing. This will establish the position of the footing onto seal.

17. Measure height of seal from mudline at four corners and record (report drawings are made from this information).

Inspect for condition of concrete

18. Inspect for soundness and appearance and take photographs when possible.

19. Inspect for spalls; measure their width, length, and height; and locate them on layout drawing.

20. Inspect for cracks and measure size, length, and depth. Record crack location at designated corrugation number by recording the distance from top of seal to the crack within that corrugation. If crack runs from one corrugation to another, record all new crack data to correspond with a different corrugation number.

21. Use a surveyor's chain for probing to determine an approximate crack depth.

22. A final inspection will show crack sizes, lengths, depths, and locations in each corrugation number on all sides of seal. Scale crack depths on the plan view for the report to show relation to footing.

Footings

Spread footings

23. Inspect for scour near footing upstream or adjacent to footing, measure size of scour (width × length × depth), and document location.

24. Inspect for scour or soft material under footing. Survey perimeter of footing. Use probing rod and rule. Station footing from upstream end to

downstream end at 2-in. increments. At these stations measure water depth, height from bottom of footing to mudline, and depth of scour from edge of footing to point under footing where bearing is established.

25. Take photographs of bottom of footing showing scour at each station when possible. Measure from top of footing to waterline on upstream and downstream ends. Measure size of footing.

26. Inspect for condition of concrete by measuring size of spalls (width × length × depth) and locations and sizes of cracks. Inspect for any exposed reinforcing steel and for soundness and appearance.

Inspect footings keyed into rock

27. Inspect for separation at base of footing and rock foundation. This condition could indicate foundation or substructure movement.

28. Inspect for voids between footing and rock foundations that could have been formed by trapped clay, silt, loose rock, or mud in concrete.

29. Inspect footings on seals for separation at base of footing and seal.

Pile footings

30. Measure width, length, and height of footing, if unknown. Measure size, number, and spacing of piles under footing, if unknown.

31. Inspect for scour at piling and record approximate depth.

32. Inspect condition of concrete at sides, top, and bottom of footing. Measure size of spalls (width × length × depth) and crack sizes and locations.

33. Inspect for exposed reinforcing steel and inspect for soundness and appearance.

34. Inspect condition of concrete where pile enters footing. Record voids or cracks.

35. Inspection of footings, if originally designed to be embedded in stream, requires measurements of each exposed pile from bottom of footing to mudline. Take photographs of bottom of footing showing exposed pile when possible.

36. Inspect piles for soundness and section loss.

37. Inspect for drift lodged between pilings.

Piles

Two divers inspecting sample pile

38. When poor visibility requires mask and light close to pile, divers can inspect on opposite sides of pile. After descending to mudline, divers can rotate to uninspected sides and ascend.

Divers inspecting piles adjacent to each other

39. Divers can choose to inspect one pile each as long as piles are adjacent. This will be more desirable in strong currents, when piles have areas to be cleaned, or in times of good visibility. Divers are to concentrate on inspecting faces of each pile.

Concrete piles

40. Inspect condition of concrete by taking photographs when possible. Measure width, length, and height of spalls. Inspect for exposed reinforcing steel or cables and for soundness and appearance.

41. Inspect for cracks and measure size, length, and location for future inspections. If a crack is spalling on each edge, record actual crack size which is measured deeper than the spalled surface. Crack length should be measured from waterline to end of crack under water. If crack extends above waterline and has not been previously recorded, measure and record. If a crack does not start at waterline, locate crack with reference to it. Bent number, pile number, and face number are required when recording location of cracks. Direction of numbering bents is from south to north or from east to west, numbering of piles is from left to right, and numbering of pile faces is counterclockwise. Take close-up photographs of cracks when possible.

42. Inspect for scour at base of piling and record approximate depth.

43. Inspect concrete when marine growth covers pile. Clean random areas of pile from waterline to mudline. The number of areas will depend on condition of concrete, visibility, water depth, and type of growth. This should be determined in the field.

44. Inspect those areas that are already clean, which in most cases are located at the mudline.

Steel piles

45. Inspect condition of steel, paint, or epoxy coating for rust. Light rust is a loose rust formation staining steel or beginning to show

through paint by pitting paint surface. Moderate rust is a looser rust formation beginning to scale or flake. These areas are discernible with no appreciable loss in steel (1/16 in. and less surface pitting). Severe rust is a heavy rust scale or heavy pitting of metal surface (1/8 in. and larger pits). Section loss of steel should be recorded in this condition. For piles with severe rust, it is recommended that a complete pass up and down the piles be made to determine the worst areas of deterioration. Scrape one or two areas to determine section loss.

46. Locate those areas of section loss and record remaining thickness of flange and web. Inspect the most vulnerable area at waterline where air and moisture cause rust. Inspect at mudline where abrasion causes section loss. Take closeup photographs of some deteriorated areas when possible. Scrape off rust flakes from worst areas and bare metal. Then measure section loss with calipers and rule. Record the remaining section and locate for future inspections.

47. Inspect steel piles with marine growth by cleaning random areas of pile from waterline to mudline. The number of areas will depend on condition of steel, visibility, water depth, and type of growth. These should be determined in the field. Inspect areas at the mudline that are already clean.

48. Inspect for scour at base of piling and record depth.

49. Inspect closely for concrete deterioration where pile enters concrete jacket under water if steel pile is incased in concrete at waterline.

50. Inspect connections if angle cross bracing extends under water.

Timber piles

51. Inspect condition of timber for soundness. Core pile under water when necessary and record remaining pile section. Plug hole with treated plugs after inspection. When decay is found, remove unsound material and determine the remaining cross section.

52. Inspect for marine life attack. Wood immersed in seawater is subject to biological deterioration. A variety of marine borers can cause loss of wood volume and a corresponding decrease in strength. The two divisions of destructive organisms are the molluscan borers and the crustacean borers.

a. Molluscan borer. The teredo, commonly called the shipworm, is an internal borer that leaves little external evidence of its destructive activity. Visible evidence of infestation is found in the form of surface pinholes or tunnels exposed by the destruction or removal of surface wood. These borers attack

anywhere below the tidal zone and bore tunnels as large as 1/2 in. in diam and 30 in. long into side pilings.

(1) When alive, the only visible sign of the animal is the two slender posterior siphons that extend above the wood surface. When anything disturbs the teredo, the siphons are withdrawn inside the tunnel.

(2) Wherever damage is detected, only an extremely rough estimate of piling strength can be made. Core samples can be effective in determining the presence of teredo borers but obviously only if the inspector is lucky enough to intersect a tunnel.

(3) The inspector should list the locations and extent of damage and indicate where it is feasible to exterminate the infestation and strengthen the member, or if it is necessary to replace it immediately.

(4) Teredos generally enter poorly impregnated creosoted piles, cracks or splints, bolt holes not properly covered, uncreosoted bracing timber, the cut ends of creosoted bracing timbers, and piles.

b. Crustacean borer. The most commonly encountered crustacean borer is the limnoria or wood louse. Limnoria is a small, lobster-like animal that gouges and erodes the wood surface. It is active principally in the intertidal zone. It bores into the surface of the wood to a shallow depth.

(1) Wave action, abrasion, and floating debris break down the thin shell of wood outside the borer's tunnel and cause limnoria to burrow deeper. The continuous burrowing results in a progressive destruction of the timber pile cross section that exhibits a characteristic hourglass shape between tide levels.

(2) The remaining pile section is to be recorded at the areas of attack by limnoria. A general attack can occur over an extended surface area and produce surface erosion.

53. Inspect for large crack and splints.

54. Inspect for drift accumulation and pile damage caused by drift and debris.

55. Inspect piles protected by concrete jackets. The concrete jackets should be inspected carefully for cracks or holes that would permit entrance of marine borers.

56. Inspect for unplugged holes.

57. Inspect pile cross bracings and condition of connecting bolts.

58. Inspect for any loss in pile section due to abrasion.

Report

59. The report should include drawings of:

   a. Elevations showing dimensions and scour, cracks, unstable conditions, etc.

   b. Sections showing degree of scour, spalling, etc., in terms of mudline and waterline.

   c. Plans showing inspection area, inspected section, spacing of piles and footings, areas of damage.

60. The report should summarize inspection data by describing general overall condition and indicating the best and worst conditions found.

REMR TECHNICAL NOTE CS-ES-1.2

SONIC PULSE-ECHO SYSTEM FOR DETERMINING LENGTH AND CONDITION OF CONCRETE PILES IN SITU

PURPOSE: To provide information on a pulse-echo system for use in nondestructive testing of concrete piles.

APPLICATION: Performing rapid nondestructive tests of concrete piles in situ. Piles can be tested from the end that is accessible with a minimum of instrumentation.

ADVANTAGES: The test is nondestructive and avoids expensive and time-consuming load tests.

LIMITATIONS: Knowledge of wave propagation theory and experience with the test are necessary for proper interpretation of results. There is a 30:1 upper limit on length-to-diameter ratio of piles which may be tested and a 5:1 lower limit.

AVAILABILITY: The Waterways Experiment Station maintains the capability to perform sonic pulse-echo tests on concrete piles in situ.

COSTS: Costs will be job-specific, depending on such factors as number of piles to be tested, on-site support furnished to the surveying team, transportation of equipment and personnel, etc.

REFERENCES:

a. Development of procedures for nondestructive testing of concrete structures; feasibility of sonic pulse-echo technique. A. M. Alexander. US Army Engineer Waterways Experiment Station, Vicksburg, MS, Apr 1980. Miscellaneous Paper C-77-11, Report 2. (NTIS No. AD A085 598).

b. State-of-the-art review of acoustic evaluation techniques. T. N. Claytor, W. A. Ellingson, R. Henneke III, H. Berger. Argonne National Laboratory, Argonne, IL, Nov 1982. ANL/FE-82-23.

FIELD PERFORMANCE: The system has been successfully used in a number of applications as described in Ref a.

BACKGROUND: Measurement of the time required for a pulse of sonic energy to pass from one boundary to another and back to the original boundary is used to determine the thickness of concrete with only one accessible surface. The reflection, or echo, from the opposite boundary will occur because of the difference in impedance of the concrete compared with air, water, or soil at the reflecting surface. The impedance of a material is defined as the product of the density and propagation velocity of the disturbance in the material. This reflection technique is referred to as pulse echo.

The pulse-echo technique of measurement uses the accurate time base of an oscilloscope to measure the time required for the echo to return from a boundary to the surface where the energy was introduced by mechanical impact. A sonic pulse travels at the longitudinal wave velocity; for a long, thin member, such as a pile, the velocity is equal to $\sqrt{E/\rho}$, where E is the modulus of elasticity and ρ is the mass density.

DESCRIPTION: The components of the sonic pulse-echo system are a digital processing oscilloscope (DPO), hammer, accelerometer, camera, filter, and associated signal-conditioning equipment. In this technique, the longitudinal wave of vibration is excited, and the total mass of the pile is placed in motion at its resonant frequency. The damping of the surrounding soil limits the pile dimensions to a 30:1 length-to-diameter ratio. The waves travel at the bar velocity ($\sqrt{E/\rho}$) for long thin members. The pile should be at least 5 times longer than its diameter to keep the velocity constant. The wavelength of the input energy must be about 10 times the diameter of the pile to maintain the constant bar velocity.

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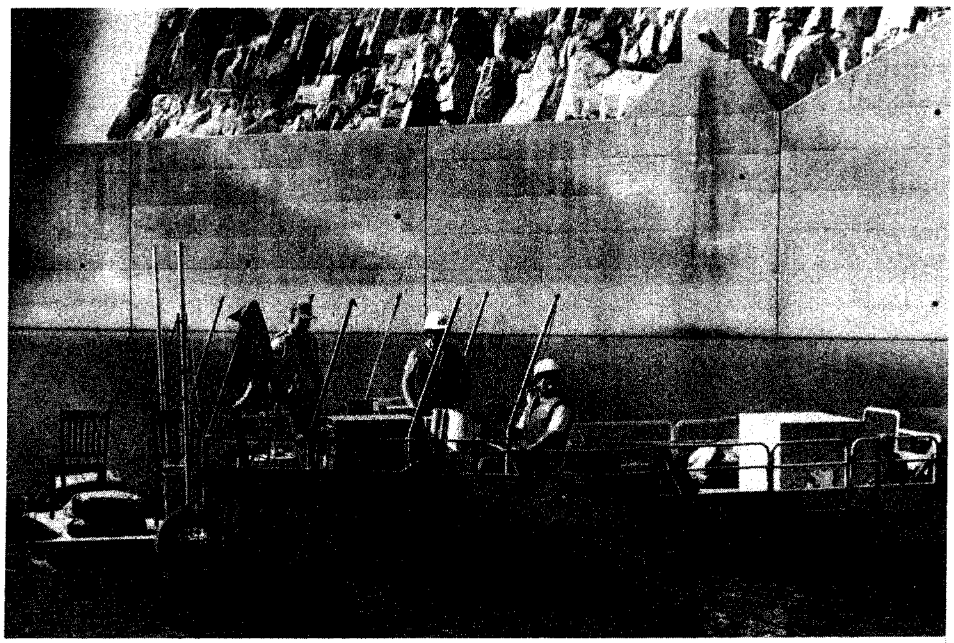
REMR TECHNICAL NOTE CS-ES-3.1

SYSTEM FOR RAPID, ACCURATE SURVEYS OF SUBMERGED HORIZONTAL SURFACES



Survey boat equipped with acoustic mapping system performing survey of stilling basin floor of Folsom Dam, a US Bureau of Reclamation project near Sacramento, CA

PURPOSE: To provide information on an acoustic mapping system that has been successfully used in underwater mapping of stilling basin floors.

APPLICATION: Performing rapid, accurate surveys of submerged horizontal surfaces such as stilling basin and lock chamber floors. The system can be used in water depths of 5 to 40 ft and produces survey results with accuracies of ± 2 in. vertically and ± 1 ft laterally.

ADVANTAGES: Avoids: (a) expense and user inconvenience associated with dewatering of structures and (b) dangers and inaccuracies inherent in diver-performed surveys.

LIMITATIONS: Use of this system is limited to a calm water environment with no significant wave action. Wave action causing a roll angle of the survey vessel of more than 5 degrees will automatically shut down the system.

AVAILABILITY: A detailed description of the system and complete specifications are available and can be furnished on request. The description and specifications can be used in preparing a Request for Proposals to conduct a survey.

COSTS: Costs will be job-specific, depending on such factors as on-site support furnished to the surveying team, transportation of equipment and personnel, etc. Estimated costs have been prepared for three sizes of jobs:

Area of Surface To Be Surveyed (sq ft)	Job Cost ($)	Job Cost ($/sq ft)
4,000	5,000	1.25
36,000	10,000	0.28
85,000	24,000	0.28

These costs include preparation of a report of the survey by the surveying team.

FIELD PERFORMANCE: The system has been successfully used in surveying the stilling basin of Folsom Dam, a US Bureau of Reclamation project near Sacramento, CA (Ref a), and the stilling basin of Ice Harbor Dam in Walla Walla District near Richland, WA (Ref b).

BACKGROUND: Erosion and downfaulting of submerged structures have always been difficult to accurately map using standard sonic surveying systems because of limitations of the systems. Side-scan sonar, fathometers, and other similar underwater mapping systems are designed primarily to see targets rising above the plane of the seafloor. Their broad sonic beams provide broad coverage, whereas a narrow beam is needed to see into depressions and close to vertical surfaces. In surveying submerged surfaces, there is also a need to know and record exactly where a mapping system is located at any instant so that defects may be precisely located and continuity maintained in repeat surveys. In order to permit surveys with the desired vertical and horizontal accuracy, as well as the required output, an acoustic mapping system was designed.

DESCRIPTION: The acoustic mapping system has three subsystems: an acoustic subsystem, a positioning subsystem, and a compute-and-record subsystem. The acoustic subsystem includes a transducer array bar having 10 transducers and a transceiver-signal processing module. The functions of the acoustic subsystem are to generate pulses to activate each of the transducers in the array bar; to amplify, rectify, and detect the reflected acoustic signal received at the transducer array; to determine the time-of-flight for the acoustic signal from the transducer to the bottom surface and back; and to output the time-of-flight data to a computer. The computer then calculates the elevation of the bottom surface using the time-of-flight information and prerecorded water level data. This information is displayed on a video terminal on board the survey boat, and the basic data are recorded on magnetic disks. The primary interrogation transducers are designed to have a narrow cone transmission pattern. Their nominal operational frequency is 360 kHz, and the narrow pattern is achieved by using a piezoelectric ceramic element whose diameter equals several wavelengths. The resultant cone dispersion is 1 degree at 6 decibels. The transducers are

classified as flat-piston radiators and transmit an essentially flat acoustic wave front. Their narrow beam design provides the capability for looking into a depression of 2 ft or larger in diameter and detecting bottom elevation to within ± 2 in. During a survey, as the boat-mounted array is moved forward, the multiple transducers are sequentially pulsed. The first signal returning to each transducer from the bottom is detected, and depth and location data are recorded.

The positioning subsystem keeps track of the position of the transducer array bar. Since the array bar is mounted on a floating work platform, it is free to move in any direction and rotate about any of three orthogonal axes; i.e., it has six degrees of freedom. The positioning subsystem is capable of controlling or determining the displacement of the bar in each of these six degrees of freedom. Because the system has excellent vertical resolution, the lateral position of the survey boat must be determined with better accuracy than the 10- to 15-ft accuracy of standard ocean surveys to take full advantage of the system capabilities. The lateral positioning network consists of a sonic transmitter on the survey boat and two or more transponders in the water at known or surveyed locations. Boat position can be calculated from two known distances. As each transponder receives the sonic pulse from the transmitter, it radios the time-of-detection back to the survey boat. Distance from the boat to the transponder can be calculated from the time-of-flight, and the position is calculated and displayed by an onboard computer. The network can be easily re-established for subsequent surveys, and the survey boat and transducer array bar can be returned to any location within the network with the same accuracy.

The compute-and-record subsystem provides for computer-controlled operation of the system and for processing, display, and storage of data. Survey results are in the form of real-time strip charts showing the absolute relief for each run, three-dimensional surface relief plots showing composite data from the survey runs in each area, contour maps of selected areas, and data printouts of the individual data point values.

REFERENCES: a. Final report: high resolution acoustic survey, Folsom Dam Stilling Basin floor. Prepared by SONEX, LTD., Richland, WA, for US Army Engineer Waterways Experiment Station, Vicksburg, MS, under Purchase Order DACW39-83-M-4340, Jan 1984.

b. Sonic inspection of Ice Harbor Dam Spillway Stilling Basin. Prepared by SONEX, LTD., Richland, WA, for US Army Corps of Engineers, Walla Walla District, Walla Walla, WA, under Contract DACW39-83-M-3397, Sep 1983, revised Feb 1984.

c. Corps-BuRec effort results in high-resolution acoustic mapping system. H. T. Thornton, Jr. In: The REMR Bulletin, Vol 2, No. 1, Mar 1985, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

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APPENDIX D

REMR TECHNICAL NOTE CS-ES-3.2

UNDERWATER CAMERA FOR INSPECTION OF STRUCTURES IN TURBID WATER

PURPOSE: To describe the components and performance of an underwater camera for use in inspection of submerged structures.

APPLICATION: Enables divers to obtain clear photographs under typical conditions encountered in inspecting underwater structural components.

ADVANTAGES:

a. Camera performs underwater.

b. Camera with all other components is maneuverable under typical inspection conditions.

c. Operation of camera is not complicated.

d. Required equipment is inexpensive.

e. Camera is capable of producing photographs in near zero visibility.

f. Photographs obtained cover an area large enough to allow evaluation of a structure.

LIMITATIONS:

a. Care must be taken to avoid the possibility of water leakage and moisture buildup inside the camera housing.

b. Care must be taken to ensure that the incidence of battery failure is minimized.

c. To ensure good contrast, the marine growth should be scraped away from the structure prior to photographing, and the correct film must be selected for the intended task.

PERSONNEL REQUIREMENTS: An experienced diver and necessary support personnel are required.

EQUIPMENT AND AVAILABILITY: Components of the system are as follows:

a. Camera--Minolta SRT 201, 35-mm.

b. Lens--28-mm and No. 1 and 3 close-ups.

c. Housing--Ikelite underwater housing with either dome or flat port.*

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* Available from Ikelite Underwater Systems, 3303 North Illinois St., PO Box 88100, Indianapolis, IN 46208; phone 317-923-4523.

d. Light source--Ikelite Substrobe M (available from same source as c).

e. Power source--rechargeable AA nickel-cadmium batteries.

f. Water box--CressiSub Batiscopia (modified with capped filling spout, light port, plastic lenses, and small-diameter end cut to accommodate housing port).** Small diameter, 6 in.; large diameter, 9-1/2 in.; approximate length, 9-1/4 in.

The camera and batteries are available on the open market.

COSTS: The entire system costs approximately $600.00.

REFERENCES: a. Underwater photography for bridge inspection. D. McGeehan. Virginia Highway and Transporation Council, Charlottesville, VA, Jul 1983. VHTRC 84-RC.

FIELD PERFORMANCE: Sites judged to be typical in terms of water current, visibility, and temperature were sought for tests, and the aid of a number of experienced divers was solicited in selecting them.

Photographs taken under typical conditions gave good detail. A photograph was successfully taken of a concrete plate with a 2-in. cross inscribed on the target. However, several photographs taken in this series were completely unusable due to mud settling on the lens. Care must be taken to allow time for any mud that might be stirred up to settle before photographing.

During these tests, an inspection sampling pattern was developed that would provide a correlation between the photographs taken and the areas of the structures they represented. Using this procedure, a diver can make successive trips to a site to perform follow-up inspections or maintenance operations. The procedure for outlining a sampling pattern and making an initial evaluation of a structure using black and white film and a final evaluation and documentation using color film is discussed in Ref a.

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** Available from Cress-Sub, 677 SW First St., Miami, FL 33130; phone 305-545-9000.